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LEADING CONTENTS

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A

Determining the cover of reinforcement, or the position of prestressing cables, immediately the concrete is placed avoids risk of unsafe structures . . .

The following is an extract from a letter received from a user of the C. & C.A. Covermeter regarding the illustration shown above—"I enclose a print of a reinforced concrete cantilever step that failed due to incorrect positioning of the reinforcing steel. Being one of a long double-flight staircase, concern was felt for the safety of the other steps. The Covermeter was used, and they were all found to be unsafe in various degrees, necessitating cutting out and renewing. The new steps were checked by Covermeter before being built in."

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The Cement and Concrete Association Covermeter, illustrated above, has been specially produced to provide an easy, accurate, and economical means of finding the exact cover of reinforcement bars, or the position of prestressing cables, without interfering with the concrete and before the concrete has hardened. It thus enables Reinforced Concrete Engineers to make sure that the specified cover of reinforcement is maintained throughout the structure, and that the position of prestressing cables is in accordance with the design. It can also be used to find the position of reinforcement, gas pipes, and other fittings in existing structures. The C. & C.A. Covermeter can be supplied for Metric or Imperial systems of measurements. For full details and prices, send to the manufacturers :

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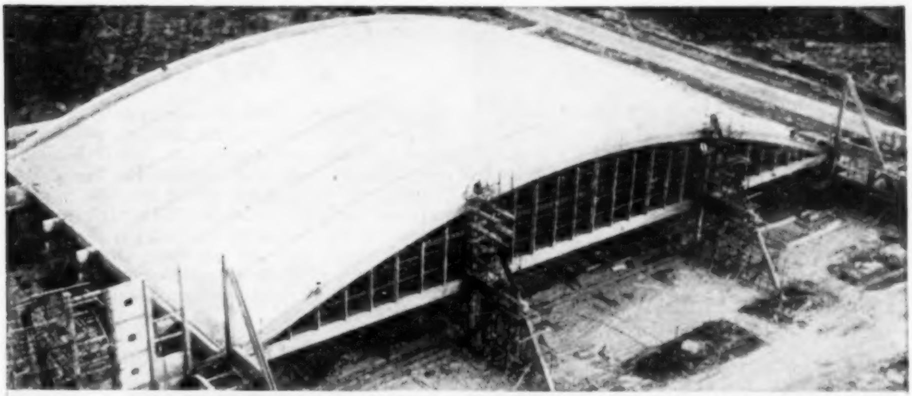
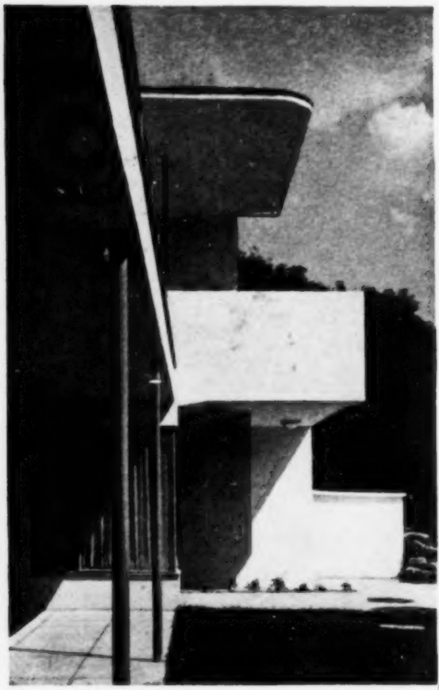




C

CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume XLVIII, No. 4. LONDON, APRIL, 1953.

EDITORIAL NOTES

Shearing Stresses in Reinforced Concrete.

AN aspect of the design of reinforced concrete that is still the subject of debate concerns its resistance to shearing forces. This is not surprising in the case of a material which is so unlike the ideal elastic materials considered in the theory of elasticity and which, in a member designed to resist bending and subjected to its working load, has already cracked over a portion of its depth. For many years the subject was confused also by doubts whether failure was more likely to be caused by diagonal tensile stresses than by vertical or horizontal shearing stresses. An interesting historical study, recently published in the U.S.A.*, of the various theories put forward since that by W. Ritter in the year 1899 shows how varied have been the views expressed.

The principles set out by Ritter for the design of web reinforcement are basically those in use to-day, but many years elapsed before his views were generally accepted ; in fact many engineers of the day were of opinion that the action of vertical stirrups is similar to that of rivets in plate-girders and serve only to transfer horizontal shearing forces. The conception of stirrups acting as dowels was shown to be erroneous by Mörsch in the year 1906, but nevertheless it is still occasionally held to-day. In general the regulations at present in use in most countries are similar and the differences are largely on the question of whether or not the tensile strength of the concrete can be considered to contribute to the strength of the member beyond the limit at which reinforcement to resist shearing forces is required. The regulations and codes of practice in this country and in other European countries do not permit reinforcement to be designed to resist only the portion of the shearing force in excess of that resisted by the concrete, as do the regulations in the U.S.A. That authorities in different countries, using the same basis for assessing the maximum shearing stresses, using similar working stresses in the concrete and steel, and using about the same factors of safety, should require varying amounts of reinforcement shows the inadequacy of our knowledge of the magnitude and distribution of stresses due to shearing forces. It is particularly in the design of members which are deep in relation to their span, or on which the loads are adjacent to the supports, that improvements of the commonly-applied methods are desirable. Although the principal tensile stress is accepted as the critical stress, little account is generally taken of the effect of the vertical compressive stresses which occur near the supports

* "What do we know about diagonal tension and web reinforcement in reinforced concrete?" E. Hognestad. University of Illinois Engineering Experiment Station. Circular Series No. 64.

when, as is usual, a load is applied to the top of a beam. With heavily-loaded short members these stresses are not negligible and, while their exact computation is not possible in the present state of knowledge, it should be possible for designers to make a judicious allowance for their effect; such an allowance would in all probability be no more inexact than other attempts to apply the theory of elasticity to reinforced concrete. Methods have, in fact, already been suggested by Continental engineers and deserve greater consideration in this country than they appear to have had.

The same considerations apply to the design of members in frames where high shearing forces are frequently accompanied by axial forces, an outstanding example being the Vierendeel girder. There has not, to our knowledge, yet been presented a generally-accepted method of calculating the stresses due to a combination of shearing forces and direct tensile or compressive forces, although it is obvious that in the first case the principal stresses will be much higher than the shearing stresses, and in the second case much less.

Although experienced engineers have little difficulty in designing reinforcement to resist shearing forces, others are apt to find the recommendations of text-books if not contradictory at least confusing. For example, the length of a beam over which a bar bent up at 45 deg. is assumed to be effective is given variously as the horizontal projection of three-quarters of the length of the sloping portion of the bar to nearly twice the effective depth of the beam.

Methods have been developed to assess the ultimate strength of beams where failure is due to bending, but where failure is primarily due to shearing forces the same degree of accuracy is not yet possible. The factors which affect the ultimate strength in shear have been shown to include the tensile strength of the concrete, the amount of longitudinal reinforcement, the amount and distribution of shear reinforcement, the ratio of the span to the effective depth of the member, and the ratio of the effective depth to the distance of the load from the support. Research on the ultimate strength of beams failing in shear appears generally to have been confined to beams in which failure by bending has been prevented by the use of large amounts of longitudinal reinforcement, or beams with thin webs, or small ratios of span to depth, or such a combination of these factors that the beams have not been representative of those commonly used in construction. Comparison of the expressions derived by various investigators for the ultimate strength in shear is very difficult because all the considerations in one series of tests frequently do not appear in another. This, to a large measure, is due to the number of factors involved, and there is likely to be much confusion of thought and wrong interpretation of the results if all these factors are not borne in mind when comparing published test results.

Obviously much experimental work remains to be done before a reliable method will be available of calculating the ultimate strength of reinforced concrete beams in shear, and particularly in members subjected simultaneously to forces producing bending, shearing, and direct stresses. This is important not only because of its application to "ultimate load" methods of design, which many engineers consider have not yet progressed sufficiently to take the place of the methods now commonly in use, but because reinforced concrete has so little in common with the ideal elastic material of theory, both methods derive their validity from tests and from the performance of actual structures.

Continuous Beams on Wide Supports.

By ALBIN CHRONOWICZ.

THE analysis of continuous beams cast monolithically with wide supports is of special significance since the British Standard Code, CP.114, permits the width of the support to be taken into account in the calculation of the bending moments. The analysis can be based on column analogy if the conception of kern points is introduced. It is therefore necessary first to define these points and consider their positions in a short eccentrically-loaded column.

Kern Points and Kern Distances.

If a point C (*Fig.* 1*a*) is at such a distance from the centroidal axis of a section of a short column that there is no stress f_b at the opposite edge of the section when a load P acts at C, the eccentricity e of the load is called a kern distance k_b, and

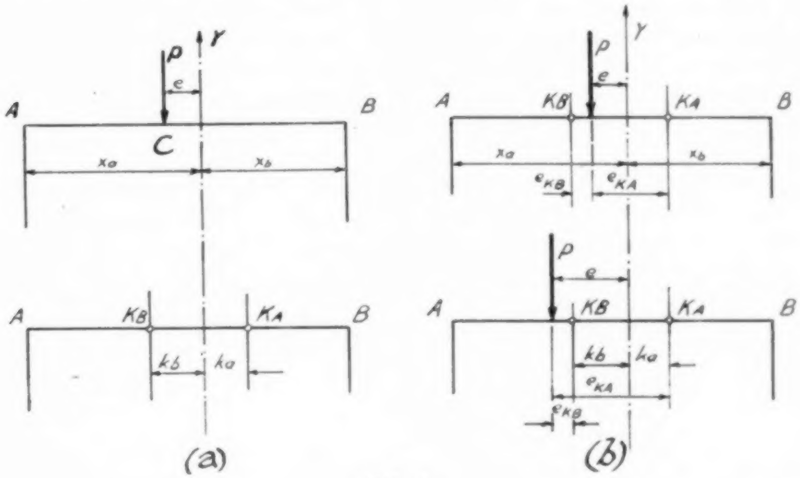


Fig. 1.

the point C is called one of the kern points K_B of the section. There is a similar kern distance k_a and kern point K_A on the other side of the centroidal axis.

If A is the cross-sectional area of the section, I the moment of inertia, ρ the radius of gyration, and x_a and x_b the distances from the centroidal axis to edges A and B respectively, the stress at B is given by

$$f_b = \frac{P}{A} - \frac{Pe}{I}x_b = \frac{Px_b}{\rho^2 A}\left[\frac{\rho^2}{x_b} - e\right].$$

It is evident that $f_b = 0$ when $e = \frac{\rho^2}{x_b}$ and, from the definition, $k_b = \frac{\rho^2}{x_b}$ and $k_a = \frac{\rho^2}{x_a}$. These kern distances are constant for a given section and do not depend on the nature of the loading.

Stresses for any Loading.

If f_a and f_b are respectively the maximum and minimum stresses produced by a load acting at an eccentricity of e towards A,

$$f_a = \frac{Px_a}{\rho^2 A}\left(\frac{\rho^2}{x_a} + e\right) \text{ and } f_b = \frac{Px_b}{\rho^2 A}\left(\frac{\rho^2}{x_b} - e\right),$$

or

$$f_a = \frac{P(k_a + e)}{I}x_a \text{ and } f_b = \frac{P[k_b - e]}{I}x_b.$$

Now $k_a + e$ and $k_b - e$ are the eccentricities of the load about the kern points K_A and K_B respectively. Denoting these eccentricities by e_{KA} and e_{KB} respectively (*Fig.* 1*b*), the products Pe_{KA} and Pe_{KB} are the moments M_{KA} and M_{KB} of the load about the kern points. Hence the stresses f_a and f_b can be expressed by

$$f_a = \frac{M_{KA}x_a}{I}; \quad f_b = \frac{M_{KB}x_b}{I}.$$

This property of the kern points leads to a straightforward determination of stiffness, carry-over factors, and fixed-end moments for use in the moment-distribution method of analysis of continuous non-prismatic beams. The application is useful in the case of beams cast monolithically with wide supports, and it leads to a fairly close approximation for beams with haunches.

Beam Constants.

The stiffness S_A at A of a beam AB of uniform cross section and fixed at B (*Fig.* 2*a*) is the moment producing unit rotation of the beam at A. Therefore S_A is $\frac{4EI}{l}$, or $4K$ if K denotes $\frac{EI}{l}$, the flexural rigidity of the beam. If the end B is freely supported (*Fig.* 2*b*), S_A is $\frac{3EI}{l}$, that is $3K$.

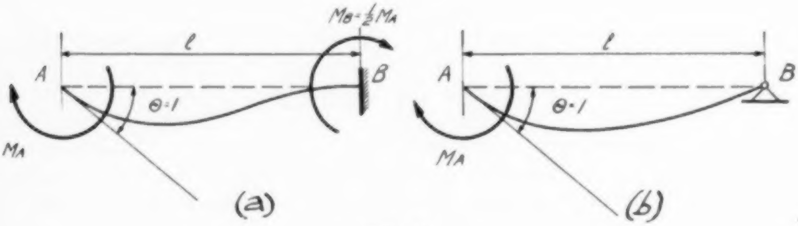


Fig. 2.

The stiffness of the beam at A can also be defined as the stress, at the corresponding edge A in the analogous column, caused by the unit load at A. For a beam fixed at both ends (*Fig.* 3*a*), the properties of the analogous column are :

Area : $A = \frac{l}{EI}$.

Moment of inertia about the centroidal axis y: $I_y = \frac{l^3}{12EI}$.

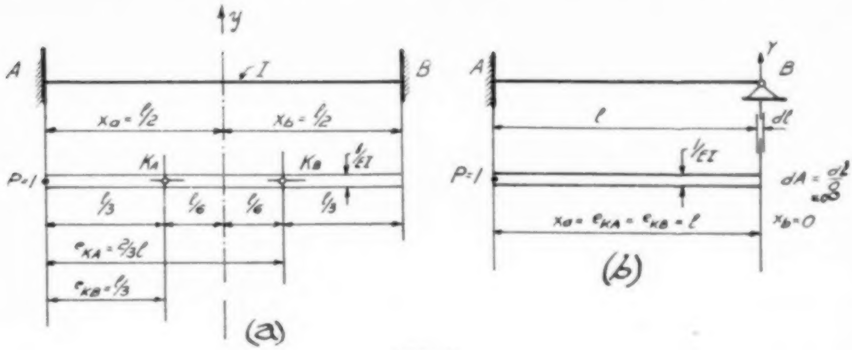


Fig. 3.

Centroidal distance: $x_a = x_b = x = \frac{l}{2}$.

Radius of gyration: $\rho^2 = \frac{I_y}{A} = \frac{l^2}{12}$.

Kern distance: $k_a = k_b = k = \frac{\rho^2}{x} = \frac{l}{6}$.

If unit load acts at A, $e_{KA} = \frac{2}{3}l$, $e_{KB} = \frac{l}{3}$, and $M_{KA} = 1 \times \frac{2}{3}l = \frac{2l}{3}$. The stress at A is given by

$$f_{a(P=1)} = S_A = \frac{2l}{3} \times \frac{12EI}{l^3} \times \frac{l}{2} = \frac{4EI}{l} = 4K.$$

The carry-over factor r_a at A is the ratio of the stresses produced at B and A by unit load at A. Therefore

$$r_a = \frac{M_{KB} \times \frac{l}{2}}{I_y} \times \frac{I_y}{M_{KA} \times \frac{l}{2}} = \frac{M_{KB}}{M_{KA}} = \frac{e_{KB}}{e_{KA}} = \frac{l}{3} \times \frac{3}{2l} = \frac{1}{2}.$$

For a beam freely supported at B (*Fig.* 3*b*) there is an infinite elastic area at that point in the analogous column corresponding to rotational freedom. Both kern points are at B, so that $e_{KA} = e_{KB} = l$, $I_y = \frac{l^3}{3EI}$, $x_a = l$, and $x_b = 0$. Hence:

$$S_A = f_{a(P=1)} = l \times \frac{3EI}{l^3} \times l = 3\frac{EI}{l} = 3K.$$

The stress f_b at B is $l \times \frac{3EI}{l^3} \times 0 = 0$; therefore $r_a = \frac{0}{3K} = 0$.

Beam Monolithic with Wide Supports.

For a beam cast monolithically with wide supports (*Fig.* 4), the moment of inertia over the width of the support is infinity, and the area of the analogous

column corresponding to this part of the beam is zero. Therefore, if the clear span of the beam is l', the properties of the analogous column (*Fig.* 5) are:

$$A = \frac{l'}{EI};\quad I_y = \frac{(l')^3}{12EI};\quad x_a = al + \frac{l'}{2};\quad x_b = bl + \frac{l'}{2};\quad \rho^2 = \frac{(l')^3}{12EI}\cdot\frac{EI}{l'} = \frac{(l')^2}{12};$$

$$k_a = \frac{(l')^2}{12(al + l'/2)};\ \text{and}\ k_b = \frac{(l')^2}{12(bl + l'/2)}.$$

Hence, $$S_A = f_{a(P=1)} = e_{KA}\cdot\frac{12EI}{(l')^3}\cdot x_a.$$

Substituting, $e_{KA} = \alpha l$, $l' = \beta l$, and $x_a = \gamma l$, $S_A = \frac{12\alpha\gamma}{\beta^3}K$.

The corresponding stress at B is

$$f_b = e_{KB}\cdot\frac{12EI}{(l')^3}\cdot x_b = \delta l\frac{12EI}{\beta^3 l^3}\gamma' l = \frac{12\delta\gamma'}{\beta^3}\times\frac{EI}{l} = \frac{12\delta\gamma'}{\beta^3}K,$$

if $e_{KB} = \delta l$ and $x_b = \gamma' l$. Hence the carry-over factor at A is $r_a = \frac{f_b}{f_a}$, and, by substitution, $r_a = \frac{\delta\gamma'}{\alpha\gamma}$.

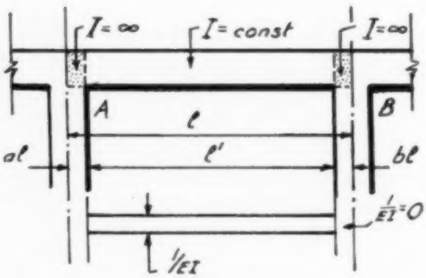


Fig. 4.

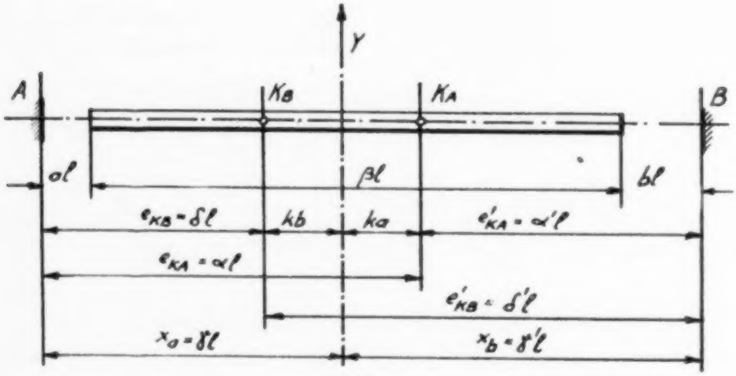


Fig. 5.

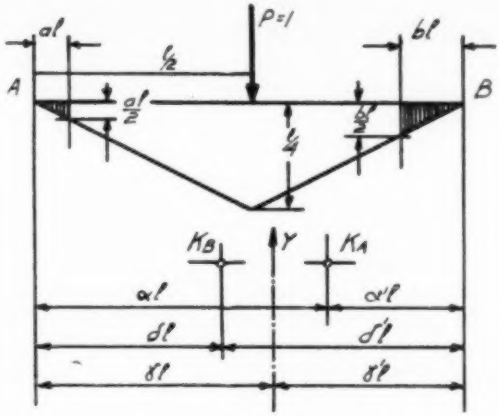


Fig. 6.

Similarly (referring to *Fig.* 5), $S_B = f_{b(P=1)} = e'_{KB}\dfrac{12EI}{(l')^3}x_b = \dfrac{12\delta'\gamma'}{\beta^3}K,$

and if $e'_{KB} = \delta' l$, $r_b = \dfrac{\alpha'\gamma}{\delta'\gamma'}$.

These formulæ are now used to calculate the fixed-end bending moments due to unit load concentrated at midspan of a beam monolithic with wide supports (*Fig.* 6). The maximum bending moment on a free span is $\dfrac{l}{4}$, and the area of the free bending-moment diagram multiplied by $\dfrac{1}{EI}$ is the analogous load. The moment of this load about the kern point K_A is

$$M_{KA} = \frac{l}{4}\cdot\frac{l}{2}\cdot\frac{1}{EI}\cdot(\alpha - 0{\cdot}5)l - \frac{al}{2}\cdot al\cdot\frac{1}{2}\cdot\frac{1}{EI}\left(\alpha - \frac{2a}{3}\right)l + \frac{bl}{2}\cdot bl\cdot\frac{1}{2}\cdot\frac{1}{EI}\cdot\left(\alpha' - \frac{2b}{3}\right)l$$

$$= \left[0{\cdot}5(\alpha - 0{\cdot}5) - a^2\left(\alpha - \frac{2a}{3}\right) + b^2\left(\alpha' - \frac{2b}{3}\right)\right]\frac{l^3}{4EI}.$$

Hence the fixed-end bending moment at A is

$$M_A^F = \left[0{\cdot}5(\alpha - 0{\cdot}5) - a^2\left(\alpha - \frac{2a}{3}\right) + b^2\left(\alpha' - \frac{2b}{3}\right)\right]\frac{l^3}{4EI}\cdot\frac{12EI}{\beta^3 l^3}\cdot\gamma l$$

$$= \left[0{\cdot}5(\alpha - 0{\cdot}5) - a^2\left(\alpha - \frac{2a}{3}\right) + b^2\left(\alpha' - \frac{2b}{3}\right)\right]\frac{3\gamma l}{\beta^3} = ml.$$

Ignoring the influence of infinite elastic areas at the intersection of a beam with wide supports, the fixed-end bending moment at A is $M_l^F = 0{\cdot}125l$. Therefore the ratio ϕ of the actual fixed-end bending moment to that based on the span between centres of the support is $\dfrac{ml}{0{\cdot}125l}$, that is $8m$, and $M_A^F = \phi M_l^F$. The general value of ϕ is given by

$$\left[0{\cdot}5(\alpha - 0{\cdot}5) - a^2\left(\alpha - \frac{2a}{3}\right) + b^2\left(\alpha' - \frac{2b}{3}\right)\right]\frac{24\gamma}{\beta^3}.$$

Omitting expressions containing the third powers of a and b,

$$\phi = (2\alpha - 1 - 4a^2\alpha + 4b^2\alpha')\frac{6\gamma}{\beta^3}.$$

Similarly $M^F_B = \phi' M^F_l$, and, omitting third powers of a and b,

$$\phi' = (2\delta' - 1 - 4b^2\delta' + 4a^2\delta)\frac{6\gamma'}{\beta^3}.$$

The factors ϕ and ϕ' can be used for other types of loading if a and b are small, since the error resulting from such approximation is generally insignificant except in the case of concentrated loads acting near the supports.

EXAMPLE (*Fig.* 7).—The elastic modulus E is omitted from the calculation as it is constant. In this example $a = 0{\cdot}05$ and $b = 0{\cdot}10$. Hence,

$$k_a = \frac{0{\cdot}85^2}{12 \times 0{\cdot}475} = 0{\cdot}127, \text{ and } k_b = \frac{0{\cdot}85^2}{12 \times 0{\cdot}525} = 0{\cdot}115.$$

Also, $\alpha = 0{\cdot}602$, $\alpha' = 0{\cdot}398$, $\beta = 0{\cdot}85$, $\gamma = 0{\cdot}475$, $\gamma' = 0{\cdot}525$, $\delta = 0{\cdot}360$, and $\delta' = 0{\cdot}640$. The stiffnesses are

$$S_A = \frac{12 \times 0{\cdot}602 \times 0{\cdot}475}{0{\cdot}85^3} = 5{\cdot}55K\,; \quad S_B = \frac{12 \times 0{\cdot}64 \times 0{\cdot}525}{0{\cdot}85^3} = 6{\cdot}55K.$$

The carry-over factors are $r_a = \dfrac{0{\cdot}36 \times 0{\cdot}525}{0{\cdot}602 \times 0{\cdot}475} = 0{\cdot}66\,;\ r_b = \dfrac{0{\cdot}398 \times 0{\cdot}475}{0{\cdot}64 \times 0{\cdot}525} = 0{\cdot}56.$

Therefore

$$\phi = [(2 \times 0{\cdot}602) - 1 - (4 \times 0{\cdot}05^2 \times 0{\cdot}602) + (4 \times 0{\cdot}1^2 \times 0{\cdot}398)]\frac{6 \times 0{\cdot}475}{0{\cdot}85^3} = 0{\cdot}99.$$

$$\phi' = [(2 \times 0{\cdot}64) - 1 - (4 \times 0{\cdot}1^2 \times 0{\cdot}64) + (4 \times 0{\cdot}05^2 \times 0{\cdot}36)]\frac{6 \times 0{\cdot}525}{0{\cdot}85^3} = 1{\cdot}32.$$

The fixed-end bending moments (for unit load on unit span) and allowing for the width of the support are as in the tabulation alongside *Fig.* 7 (ii), the exact values of fixed-end moments being given in brackets. It is left to the designer to judge if these approximations are acceptable.

When more exact values are required, the following general rule can be applied (*Fig.* 8):

The area G of the free bending-moment diagram on the clear span βl, the position of the centroid C, and the respective eccentricities e_{KA} and e_{KB} about the kern points can be determined by semi-graphical approximate methods. The fixed-end bending moments are then

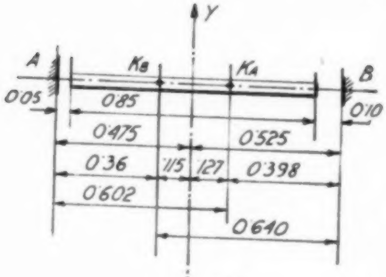
$$M^F_A = \frac{12Ge_{KA}\cdot\gamma l}{(\beta l)^3}\,; \quad M^F_B = \frac{12Ge_{KB}\cdot\gamma' l}{(\beta l)^3}.$$

Wide Support at One End only.

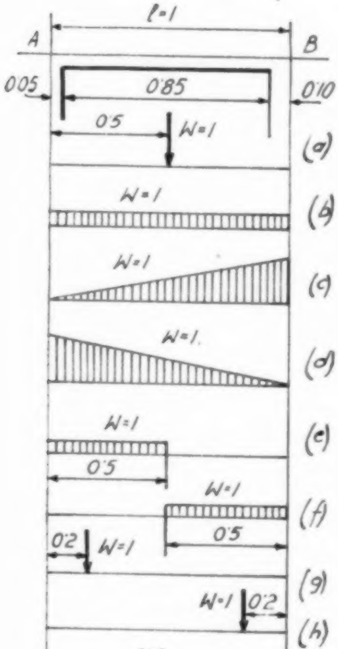
If a fixed-ended beam (*Fig.* 9) has a wide support at one end only, simplification of the beam constants results. As before,

$$A = \frac{\beta l}{I}\,; \quad I_y = \frac{\beta^3 l^3}{12I}\,; \quad \rho^2 = \frac{\beta^2 l^2}{12}\,; \quad k_a = \frac{\beta^3 l}{12\gamma}\,; \quad k_b = \frac{\beta^2 l}{12\gamma'}\,;$$

$$S_A = \frac{12\alpha\gamma}{\beta^3}K\ ;\quad S_B = \frac{12\delta'\gamma'}{\beta^3}K\ ;\quad r_a = \frac{\delta\gamma'}{\alpha\gamma}\ ;\quad \text{and } r_b = \frac{\alpha'\gamma}{\delta'\gamma'}.$$



(i)



(ii)

Load a: $M_A^F = 0{\cdot}99 \times 0{\cdot}125 = 0{\cdot}124\ (0{\cdot}122)$.

$M_B^F = 1{\cdot}32 \times 0{\cdot}125 = 0{\cdot}165\ (0{\cdot}167)$.

,, b: $M_A^F = 0{\cdot}99 \times 0{\cdot}083 = 0{\cdot}082\ (0{\cdot}082)$.

$M_B^F = 1{\cdot}32 \times 0{\cdot}083 = 0{\cdot}110\ (0.109)$.

,, c: $M_A^F = 0{\cdot}99 \times 0{\cdot}067 = 0{\cdot}066\ (0{\cdot}080)$.

$M_B^F = 1{\cdot}32 \times 0{\cdot}10 = 0{\cdot}132\ (0{\cdot}110)$.

,, d: $M_A^F = 0{\cdot}99 \times 0{\cdot}10 = 0{\cdot}099\ (0{\cdot}109)$.

$M_B^F = 1{\cdot}32 \times 0{\cdot}067 = 0{\cdot}088\ (0{\cdot}082)$.

,, e: $M_A^F = 0{\cdot}99 \times 0{\cdot}115 = 0{\cdot}114\ (0{\cdot}127)$.

$M_B^F = 1{\cdot}32 \times 0{\cdot}051 = 0{\cdot}067\ (0{\cdot}064)$.

,, f: $M_A^F = 0{\cdot}99 \times 0{\cdot}051 = 0{\cdot}050\ (0{\cdot}036)$.

$M_B^F = 1{\cdot}32 \times 0{\cdot}115 = 0{\cdot}152\ (0{\cdot}155)$.

,, g: $M_A^F = 0{\cdot}99 \times 0{\cdot}128 = 0{\cdot}127\ (0{\cdot}149)$.

$M_B^F = 1{\cdot}32 \times 0{\cdot}032 = 0{\cdot}042\ (0{\cdot}029)$.

,, h: $M_A^F = 0{\cdot}99 \times 0{\cdot}032 = 0{\cdot}032\ (0{\cdot}013)$.

$M_B^F = 1{\cdot}32 \times 0{\cdot}128 = 0{\cdot}169\ (0{\cdot}173)$.

Fig. 7.

Similar simplifications occur in the expressions for the correcting factors ϕ and ϕ'. For a central unit load (*Fig.* 10):

$$M_{KA} = 0{\cdot}25 \times 0{\cdot}5(\alpha - 0{\cdot}5) - \frac{a}{2}\cdot\frac{a}{2}\left(\alpha - \frac{2a}{3}\right) = 0{\cdot}125(\alpha - 0{\cdot}05) - \frac{a^2}{4}\left(\alpha - \frac{2a}{3}\right).$$

$$M_A^F = \left[0{\cdot}125(\alpha - 0{\cdot}5) - \frac{a^2}{4}\left(\alpha - \frac{2a}{3}\right)\right]\frac{12\gamma}{\beta^3}.$$

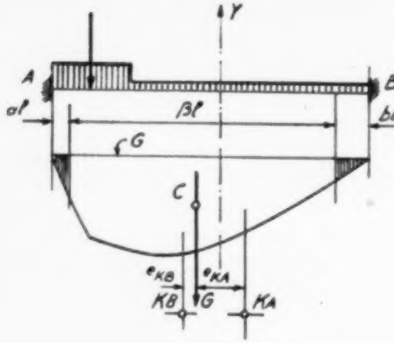


Fig. 8.

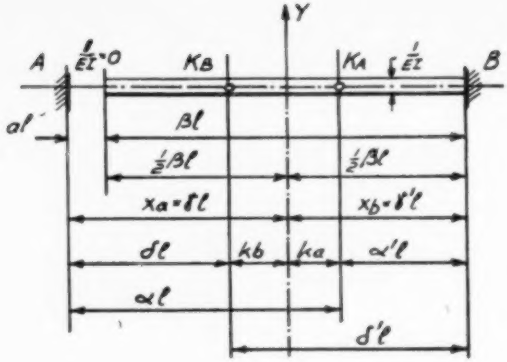


Fig. 9.

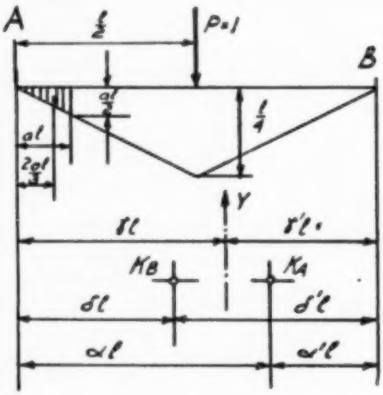


Fig. 10.

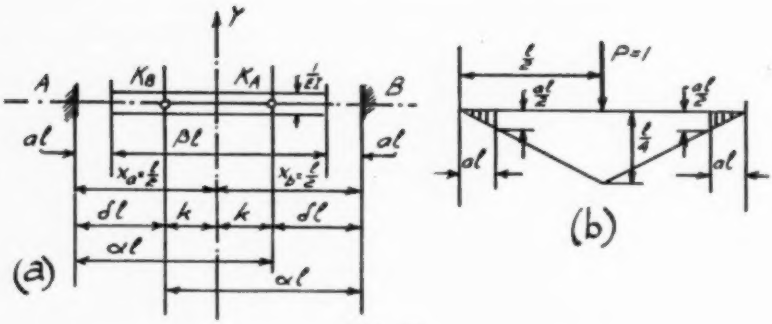


Fig. 11.

$$\phi = \left[0{\cdot}5(\alpha - 0{\cdot}5) - a^2\left(\alpha - \frac{2a}{3}\right)\right]\frac{24\gamma}{\beta^3} \simeq 6(2\alpha - 1 - 4a^2\alpha)\frac{\gamma}{\beta^3}.$$

$$M_{KB} = 0{\cdot}125(0{\cdot}5 - \delta) + \frac{a^2}{4}\left(\delta - \frac{2a}{3}\right).$$

$$M_B^F = \left[0{\cdot}5(0{\cdot}5 - \delta) + a^2\left(\delta - \frac{2a}{3}\right)\right]\frac{3\gamma'}{\beta^3}.$$

$$\phi' = \left[0{\cdot}5(0{\cdot}5 - \delta) + a^2\left(\delta - \frac{2a}{3}\right)\right]\frac{24\gamma'}{\beta^3} \simeq (1 - 2\delta + 4a^2\delta)\frac{6\gamma'}{\beta^3}.$$

Symmetrical Beams.

These expressions, when modified for symmetrical beams (*Fig.* 11), are very simple.

$$A = \frac{\beta l}{I};\ I_Y = \frac{\beta^3 l^3}{12 I};\ \rho^2 = \frac{\beta^2 l^2}{12};\ k_a = k_b = k = \frac{\beta^2 l}{6}.$$

$$S_A = S_B = S = \frac{6\alpha}{\beta^3};\ r_a = r_b = r = \frac{\delta}{\alpha}.$$

$$M_A^F = M_B^F = M^F = \frac{G}{A} = \left[(0{\cdot}25 \times 0{\cdot}5) - 2\frac{a}{2}.\frac{a}{2}\right]\frac{1}{\beta} = [0{\cdot}25 - a^2]\frac{1}{2\beta}\ (Fig.\ 11b).$$

$$\phi = (0{\cdot}25 - a^2)\frac{8}{2\beta} = \frac{(1 - 4a^2)}{\beta},\ \text{and since } \beta = 1 - 2a,\ \phi = 1 + 2a.$$

EXAMPLE (*Fig.* 12).—For unit load on unit span the fixed-end bending moments calculated from the foregoing formulæ (the exact values being given in brackets) are :

Load a : $M^F = 0{\cdot}083 \times 1{\cdot}2 = 0{\cdot}0995$ $(0{\cdot}0983)$.

,, b : $M_A^F = 0{\cdot}067 \times 1{\cdot}2 = 0{\cdot}079$ $(0{\cdot}0767)$.

$M_B^F = 0{\cdot}1 \times 1{\cdot}2 = 0{\cdot}12$ $(0{\cdot}1307)$.

The approximation $\phi = 1 + 2a$ cannot be used in exact calculations for concentrated loads near the supports, and it is advisable to determine the values of fixed-end bending moments by the method described for unsymmetrical beams.

One End of Beam Freely Supported.

If the beam is freely supported at one support (*Fig.* 13), the beam constants are $I_Y = \frac{\beta^3 l^3}{3I}$, and $S_A = l\frac{3I}{\beta^3 l^3}l = \frac{3}{\beta^3}K$. Therefore

$$M_K = \left[\left(\frac{1}{8} \times \frac{1}{2}\right) - \frac{a^2}{4}\left(1 - \frac{2a}{3}\right)\right]\frac{l^3}{I} = [3 - 4a^2(3 - 2a)]\frac{l^3}{48I}.$$

$$M^F = \frac{1}{48}[3 - 4a^2(3 - 2a)]\frac{l^3}{I}.\frac{3I}{\beta^3 l^3}.l = [3 - 4a^2(3 - 2a)]\frac{l}{16\beta^3}.$$

$$\phi = \frac{1}{16}[3 - 4a^2(3 - 2a)]\frac{l}{\beta^3}.\frac{16}{3l} = \frac{3 - 4a^2(3 - 2a)}{3\beta^3} \simeq \frac{1 - 4a^2}{\beta^3}.$$

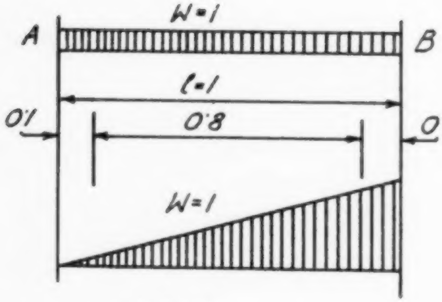


Fig. 12.

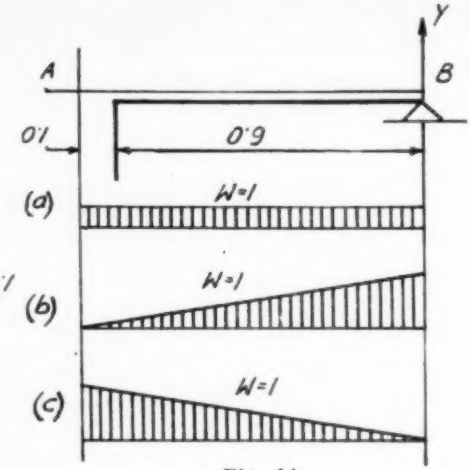


Fig. 14.

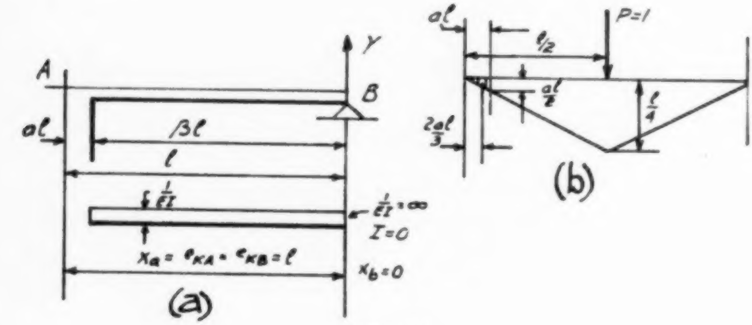


Fig. 13.

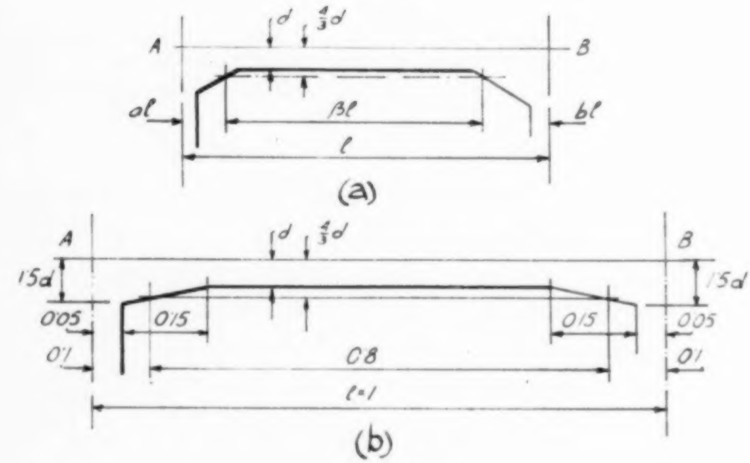


Fig. 15.

EXAMPLE (*Fig.* 14).—As before, there is unit load on unit span, and exact values of the fixed-end bending moment are given in brackets.

$$a = 0{\cdot}1\ ;\quad S = \frac{3}{0{\cdot}9^3}K = 4{\cdot}1K\ ;\quad \phi = \frac{1 - 0{\cdot}04}{0{\cdot}9^3} = 1{\cdot}32.$$

$$\text{Load } a:\ M^F = 0{\cdot}125 \times 1{\cdot}32 = 0{\cdot}165\ (0{\cdot}162).$$
$$\text{,, } b:\ M^F = 0{\cdot}116 \times 1{\cdot}32 = 0{\cdot}153\ (0{\cdot}152).$$
$$\text{,, } c:\ M^F = 0{\cdot}133 \times 1{\cdot}32 = 0{\cdot}175\ (0{\cdot}172).$$

Haunched Beams.

The method described can be readily applied to the approximate analysis of haunched beams. The "clear span", corresponding to the span between the faces of the supports, is assumed to be the distance between the points at which the depth of the haunch is $1\frac{1}{3}$ times the depth of the beam (*Fig.* 15*a*). The approximate results are given in the following example of a symmetrical beam with straight haunches (*Fig.* 15*b*). The approximate constants are:

$$k = \frac{0{\cdot}8^2}{6} = 0{\cdot}11\ ;\quad \alpha = 0{\cdot}61\ ;\quad \delta = 0{\cdot}39\ ;\quad S = 0{\cdot}61\frac{6}{0{\cdot}8^3} = 7{\cdot}2\ (7{\cdot}42)\ ;$$

$$r = \frac{0{\cdot}39}{0{\cdot}61} = 0{\cdot}64\ (0{\cdot}641)\ ;\quad \text{and } \phi = 1{\cdot}2.$$

The exact values are given in brackets. The exact value of ϕ is 1·21 for a concentrated load at midspan and 1·18 for a uniformly-distributed load.

Fire Grading of Buildings.

THE principles of the design of buildings to reduce the risk of fire and to provide protection against fire were considered in "Fire Grading of Buildings, Part I," Post-war Building Study No. 20, published in 1946. A further three parts have now been published dealing with additional precautions considered necessary to ensure the safety of the occupants of a building. These are "Fire Grading of Buildings. Parts II, III and IV," Post-war Building Study No. 29 (H.M.S.O. Price 4*s*. 6*d*.), and are concerned with planning rather than the fire-resistance of building materials.

Part II deals with fire-detection and fire-fighting equipment and includes information on the space requirements for escape ladders up to 100 ft. high. Methods and data for assessing the safe density of population of buildings used for various purposes and recommendations on the numbers and positions of exits are given in Part III, together with notes on structural precautions to reduce the spread of smoke, flames, and fire. Tables give recommendations on the number of persons to be permitted to use staircases of stated widths and from two to ten stories high for various densities of occupation. The construction of chimneys, flues, and hearths, and common defects in chimneys and flues in existing buildings, are considered in Part IV. Methods are described of insulating existing chimneys to reduce the risk of fire.

Appendixes include data on the rate of movement of people through exits, widths of staircases to serve stated numbers of persons, the supposed causes of fires, and the proposed minimum requirements for graded types of construction (reprinted from Part I).

Information relating to reinforced concrete construction was given in Part I and reviewed in this journal for June, 1947.

The Ultimate Strength of Prestressed Beams.

From Professor A. L. L. Baker.

THE article by Professor G. Magnel on "The Ultimate Strength of Prestressed Beams" in this journal for February last contains some valuable results of tests of prestressed beams and an interesting proposal for simplifying the calculation of the ultimate strength in bending of prestressed beams.

Many engineers would, however, probably like to know what was the value of the prestressing in each beam at the time of the tests, whether or not the cables were bonded, and the precise definition of failure in the "under-reinforced" beams. Is not the linear relationship shown to exist between K and λ due to failure being initiated by non-recoverable strain causing a rise of the neutral axis? Also, do not the higher values of K, say, 0·25 to 0·33, depend not only on the area of steel but also on the prestress and on the efficiency of the bond? *Fig.* 1 shows three possible distributions of strain for the same area of steel: (1) A bonded beam in which the strain in the steel due to the load at failure minus the strain due to pre-tensioning is 0·0024; (2) An end-anchored beam in which this difference in strain is 0·0024; and (3) A bonded beam in which this difference in strain is 0·0072.

The position of the neutral axis is at about half the depth of the beam in the case of (1) and at about a quarter the depth in cases (2) and (3), so that the ultimate moment of resistance of the concrete for (1) is much greater than for (2) or (3). While the formula $M = 0{\cdot}318bd^2c_{up}$ may be correct when $100\lambda = 0{\cdot}84$ and the prestressing force and the bond are adequate, it is not strictly a general formula, which should include terms expressing the influence on the ultimate strength of the prestress and of the bond. Unless the special conditions for which the formula is valid are precisely stated, there is a danger that, well known though the conditions may be to specialists in prestressed concrete, the importance of bond and the correct prestress may not be appreciated by others.

Contractors will usually take care of structural requirements which are self-evident, but it is not at all obvious that a reduced prestress or lack of bond strength may be as serious as the omission of steel or cement. It is interesting to note that in your February number you reported that one cause of cracking under full working load of the footbridge at the

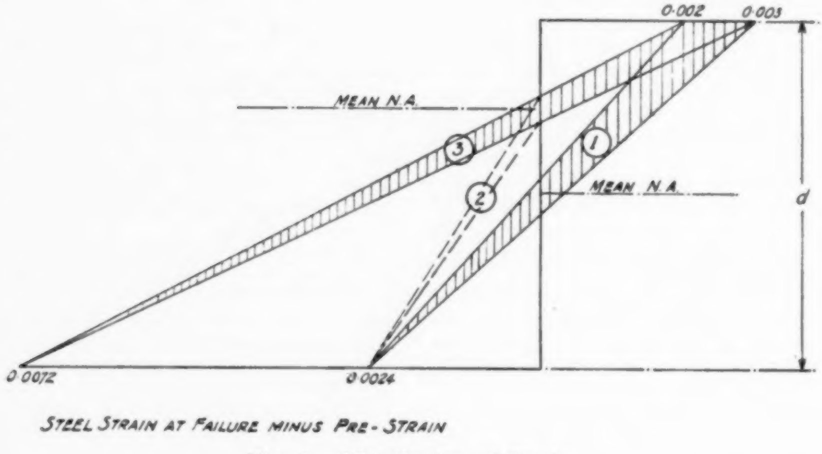


Fig. 1.—Distribution of Strain.

Festival of Britain site on the South Bank, London, was ineffective grouting.

From Mr. Donovan H. Lee, B.Sc. (Eng.), M.I.C.E.

I have little doubt all will agree that Professor Magnel has made an excellent contribution to this subject in your February number. I do not quite understand, however, why he appears to criticise some of the formulæ at present in use for calculating the ultimate strength. Professor Magnel will no doubt agree that the ultimate strength of prestressed beams has for some time been calculable by these formulæ as accurately as it is possible to calculate the ultimate strength of beams of most other structural materials.

Formula (1), $M_t = 0{\cdot}95\ A_T t_u d$, given by Professor Magnel, is limited to a restricted range of conditions, but even so it appears to give an ultimate strength slightly greater than, for example, the formula given by Dr. Abeles, $M_t = KA_t t_u ad$, since a varies between about 0·75 and 0·93 (say, 0·9 in the range named by Professor Magnel) and K is usually below 1·0 except when using pretensioned wires.

In other words, formula (1) if within about 10 per cent. of the ultimate strength would, I think, generally give values on the upper limit of this margin. Although in a test of one of the beams of the Tampa Bay bridge in which high-tensile bars were used the value K in Dr. Abeles' formula is greater than 1·0, in this case a has a value of about 0·95 and so fits in well with Professor Magnel's formula. The test was made with the deck slab acting in conjunction with the beam and therefore the value of 0·95 was practically ensured. The test would, perhaps, at least confirm that the bond was adequate to develop the full strength. The anchorage between the slab and the top of the beam was, as usual, excellent.

Whether or not formula (1) gives the average ultimate strength within 10 per cent. as suggested by Professor Magnel for the conditions named by him, or whether that formula gives values close to the upper limit, it would seem that the other formula I have mentioned for the ultimate strength will be more accurate unless K and a are both near to 1·0.

Undoubtedly Professor Magnel is correct in pointing out that some variations must be expected between the ultimate strength as calculated and that obtained by test. Variation in the stress-strain curve for the wires between the 0·2 per cent. proof stress and the ultimate strength is, I think, of more importance than has been realised.

Bond has also been frequently pointed out as affecting the results, greasy or very smooth wires giving, of course, lower strengths. With bars the increased bond stress is counterbalanced by the very small quantity of grout needed, which reduces the loss of bond due to shrinking of the grout away from the surface of the concrete in the cavity. If the same care is taken in grouting bars and wires then either formula applies equally well whether the beam contains bars or wires, although for loads greater than the load causing cracking the deflections should be less in beams with bars.

Partially-prestressed beams (although the load causing cracks in them is lower and the deflection greater) have virtually the same ultimate strength as fully-prestressed beams. Also reinforced concrete, particularly when using cold worked high-tensile bars with deformed surfaces, has the same ultimate strength although with larger deflections.

It would appear, therefore, that Professor Magnel's formula (1) could easily be given wider application by variation of the coefficient of 0·95, but this would seem to bring the formula into line with other formulæ already available except for the question of whether 0·95 in Professor Magnel's formula (1) is not higher than the mean value for normal fully-bonded construction. I would, therefore, be interested to know whether Professor Magnel considers that the examples on which his formula (1) is based represent the most favourable cases of effectiveness of bond or average cases. Alternatively if formula (1) is correct for the average case of post-stressed beams with well grouted wires, Dr. Abeles' formula must be pessimistic if a value of K below 1·0 is used for normal grouted conditions.

From Professor G. Magnel.

With regard to the comments by Professor Baker, I considered only anchored cables or cables in which wires with increased bond are used so that no slipping of the wires occurs before failure of the beam at midspan due to the ultimate

strength of either the concrete or the steel being exceeded. In my opinion the amount of the initial prestress is of no importance whatever in connection with the ultimate resistance of a beam with wires that do not slip. This is because the strain in the steel due to prestressing is negligible in comparison with the strain at failure. The results of the tests on the beam reinforced with Neptune steel confirm this (see pages 76 and 77 of this journal for February last).

So little is known of the strain of concrete at failure that I do not wish to make any comment on Professor Baker's statements regarding the strain diagrams. Some writers say that the strain at failure is constant, but the values given vary by 100 per cent. according to the writer.

The only way to show that my proposed formula is unsatisfactory is to compare it with further tests in which the results deviate by more than ten per cent. from my straight-line diagram of K_1 as a function of λ. In this it is necessary to take into account the range of applicability of the formula [see items (*a*) to (*h*) on pp. 74 and 75 of my article].

I wish to stress the fact that my intention was not to discuss the problem in relation to small factory-made beams with pre-tensioned wires, as my interest in prestressed concrete has always been in larger engineering works.

Regarding the comments of Mr. Donovan Lee, I did not in my article criticise any other formulæ but only the way in which some writers produce these formulæ from what I consider to be questionable theories, in which too many assumptions are made regarding the stress-strain diagrams of steel and concrete. My formula is based on the results of tests on beams stressed by post-tensioned and anchored cables, except in one case where I considered a beam which is not prestressed but reinforced with high-tensile steel with a greatly increased bond strength. I have never attempted to compare my formulæ with the results of tests on beams prestressed by bonded wires without anchors. This case is more complicated if only because the wires are generally distributed over a greater area of the cross section of the beam.

My formula applies equally well to beams in which failure occurs first in the steel as it does to beams in which the concrete fails first. The formula which interests me most is formula (2), which is applicable even in cases where the area of steel is exceptionally great and where the crushing strength of the concrete has no practical importance even when the beam fails by crushing of the top flange.

Proposed Terminal Building at Renfrew Airport.

THE proposed Terminal Building at Renfrew, the airport for Glasgow, is shown in *Fig.* 1. The main beams in the roof of the concourse are supported at the front of the building by hangers from a reinforced concrete arch 48 ft. high. The buildings have been designed to permit extensions to be made to them.

The architect is Mr. W. H. Kininmonth, A.R.S.A., F.R.I.B.A., of Messrs. Rowand Anderson, Kininmonth & Paul, and the consulting civil engineers are Messrs. Blyth & Blyth. The contract has been let by the Ministry of Civil Aviation to Messrs. A. A. Stuart & Sons (Glasgow), Ltd., and work on the site has started. It is intended that the new building shall be completed by the summer of 1954.

Fig. 1.

Foundation for a Large Hammer.

PRESTRESSED GROUTED CONCRETE.

By ROLT HAMMOND, A.C.G.I., A.M.I.C.E.

THE forging-hammer foundation described in this article was designed by Mr. J. H. A. Crockett, B.Sc. (Eng.), A.M.I.C.E., for the International Nickel Company of Canada, at Huntington, West Virginia, U.S.A. The hammer is employed for the hot forging of nickel-alloy ingots and sometimes for the hot-cold working of nickel and nickel-copper blooms and billets, using superheated steam at high pressure and therefore imparting the greatest shock experienced in general forging practice. The hammer has an 8-tons tup and the foundation has been analysed for shock, vibrations, movements, and loads in all directions. The inertia blocks were prestressed in three directions simultaneously on the Freyssinet system.

The main requirement was that there must be no large ground vibrations. Consideration was given to the shock on the foundation at the moment of impact; the vibrations, loads, and movements in all directions due to impact; and to the fatigue of the concrete caused by the rapid reversals of stress. The arrangement is shown in *Fig.* 1. The upper and lower inertia blocks each weighs about 160 tons. The upper block is annular and mounted on four massive concrete legs which rest on the corners of the lower block so as to provide access to the laterally-disposed springs and other dampers and to the sides of the anvil-block and its springs. The sideways-acting rubber springs are on welded steel brackets, fixed to the lower block by high-tensile steel bolts which were prestressed by the Lee-McCall system. These bolts were tensioned to 30 tons per square inch, by jacks attached to their lower ends. The brackets were shimmed so that the springs have a precompression such that under oscillatory movement they will not become loose or chatter even when subjected to a heavy eccentric blow causing lateral and rotational movement of all the masses. The superstructure of the hammer and the anvil are in accurate relationship to one another, and cannot become out of true when they have been adjusted and after the initial and working creep of the rubber. The hammer, the foundation, and the adjacent ground act as a unit, and all these parts of the oscillating system are interrelated.

The length of the wave which travels through the anvil and the rest of the system is determined by the duration of the blow, the density of the material through which the wave travels, and the modulus of elasticity of the material. The length of the wave is only a few feet when the hammer is forging very hard nickel-alloy, and this very short wave creates problems of special difficulty. The wave is reflected and refracted from surfaces and boundaries in a similar manner to an extremely powerful sound wave. The shock-wave, on reaching the bottom of the anvil, is partly reflected back, but a small amount is refracted through the rubber spring into the concrete. Shock-waves in compression, when reflected from a concrete-air boundary, are in tension, and should any of these come to a focus at any point the resulting tensile stress would be sufficient to cause failure. The concrete blocks therefore have a shape which will cause the shock-waves to disperse rather than to come to a focus. Waves passing through concrete which is not uniform in quality have a greater velocity in the

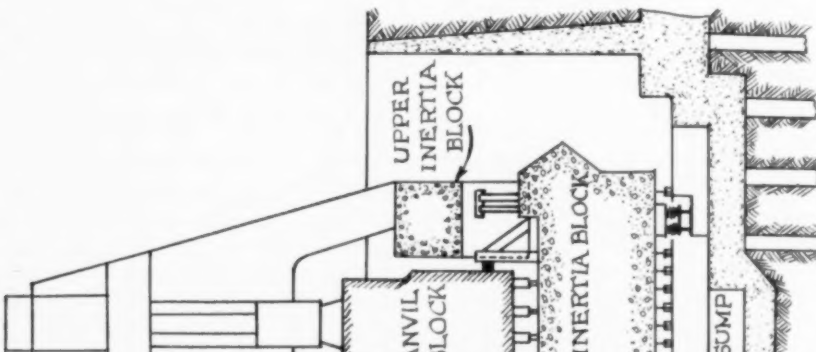


Fig. 1.—General Arrangement of Foundation.

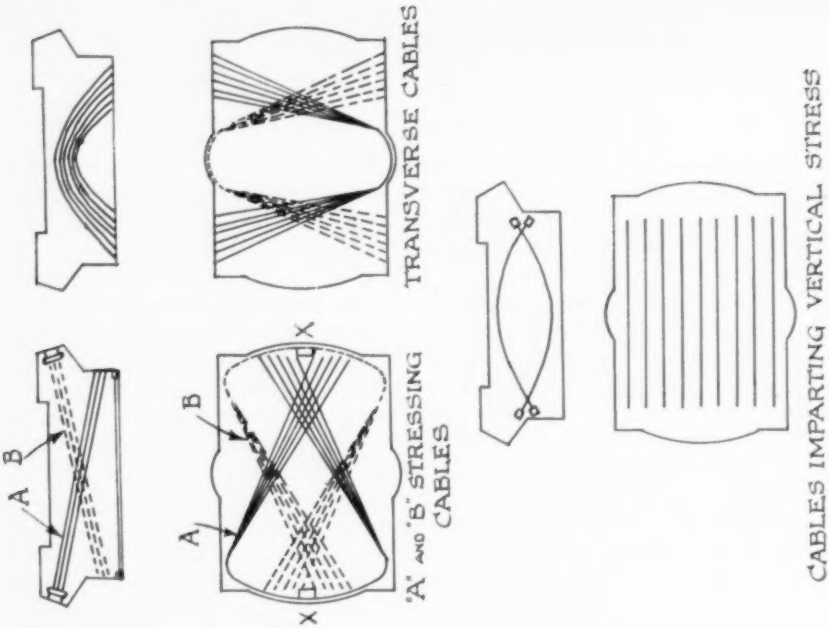


Fig. 2.—Arrangement of Cables.

better quality concrete than in the poorer, so setting up a difference in phase between the two parts of the wave. A strong shearing action is thus produced between the two which leads to disintegration. The concrete should therefore be as homogeneous as possible.

The fatigue strength of concrete is much lower when the stress is alternately tensile and compressive than when the stress varies in intensity but remains compressive. After ensuring the greatest possible isolation of the foundation the live load from the anvil was still about five times the dead load, and so it was decided to prestress the inertia blocks in three directions to maintain a state of compression within the concrete. In order not to disturb production, the existing concrete-lined foundation pit was enlarged and provided with a new bottom on the existing piles. At the level of the floor of the old pit a strong reinforced concrete frame was formed to resist the lateral earth thrust and the outward thrust due to the side springs on the lower inertia block. Cables A and B (*Fig.* 2), which impose forces along the major axis of the block, pass diagonally from one end of the block to the other, where they return through a curve of large radius to the end from which they started. By sloping these cables diagonally it was thought that they would also exert a vertical force on the block, but in addition, to increase the vertical force, further cables were placed along the major axis of the block, alternate cables being curved in opposing directions in a vertical plane. Two more sets of cables, disposed diagonally, provide the transverse forces. The cables pass through sheaths embedded in the concrete. In order to reduce losses due to friction at the bends, the cables were over-tensioned initially and during the prestressing, then allowed partly to slip back before being anchored. Cables A and B each had two bends of more than a right angle, and, as this method of tensioning would not provide uniform extension, two flat jacks were inserted at each end at the points marked X (*Fig.* 2), so that the middle of the curved portion of each cable would be pulled around the corners. *Fig.* 2 refers to the lower inertia block, and the cables are arranged in a similar manner in the upper block.

The upper block has four strongly-reinforced concrete feet which rest partly on the overhanging parts of the lower block ; for this reason the double-curved cables were placed around the corners of the lower block. Although the prestressing forces are sufficient to resist the calculated bending moments, mild steel bars were placed near the surfaces of each block in order to prevent development of fine hair cracks which might lead to failure as a result of fatigue. It was also thought that the cables might slip through the anchor-cones owing to the constant fluctuation of the dynamic stresses. For this reason, after tensioning, the wires forming the cables were threaded through a ⅜-in. thick mild steel plate placed outside the male cones, and these plates, together with the cables, were grouted.

The aggregate for the concrete in the extension to the walls and bottom of the foundation pit was crushed dolomitic limestone and crushed limestone sand, the fineness modulii being 6·70 and 2·79 respectively. The proportions were 1 : 2·01 : 2·35 by weight, and the average crushing strength of 6-in. by 12-in. cylinders was 4837 lb. per square inch at 28 days. For the upper and lower inertia blocks a strength of about 4000 lb. per square inch was required at seven days.

A crushing strength of 10,000 lb. per square inch at 28 days has been attained

on 6-in. cubes, and 13,000 lb. per square inch at 128 days. The close spacing of the reinforcement and cables required concrete which could be easily placed. A high cement content was not desirable, due to the danger of excessive shrinkage. The method of concreting was first to fill the shuttering with dry or damp coarse aggregate and then fill the voids with grout through tubes ¾-in. diameter; this is known as the "Prepakt" method. The coarse aggregate was crushed

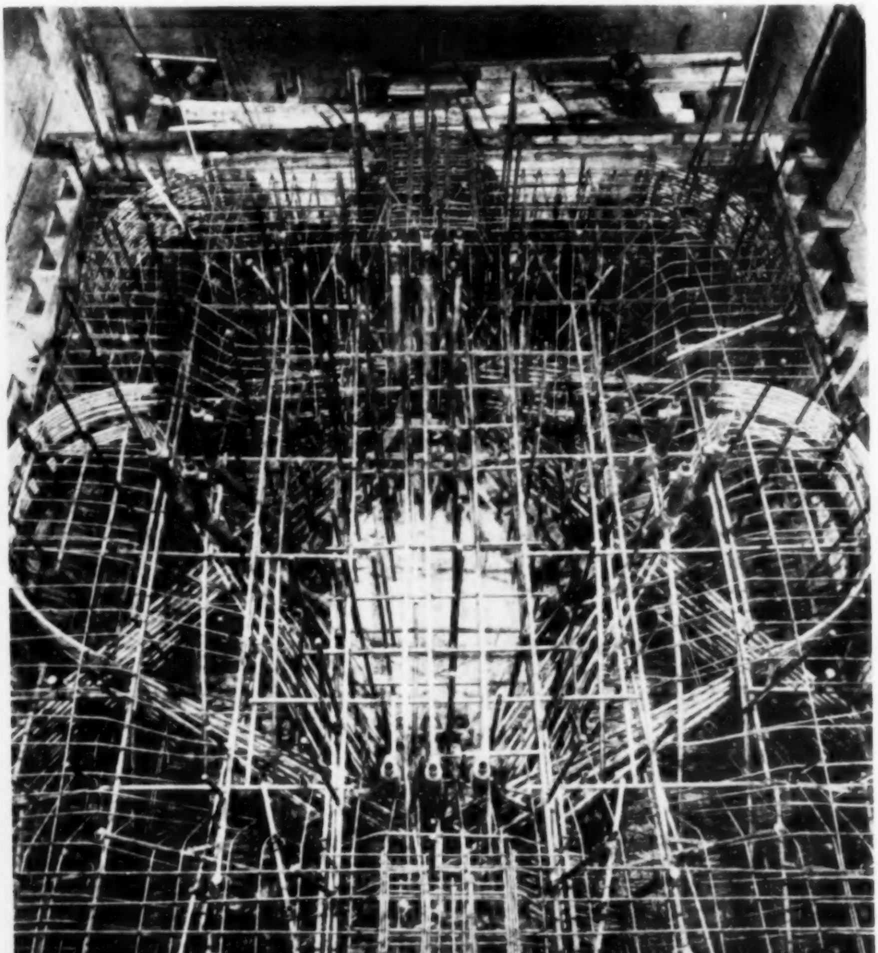


Fig. 3.—Prestressing Cables in Lower Inertia Block. Note Tops of Grouting Tubes.

limestone with a fineness modulus of 6·97. The grout was a mixture of Portland cement, finely-divided pozzolanic material, and sand in the proportions of 9 : 1 : 6. It is claimed that the pozzolana prevents agglomeration of cement particles and decreases "bleeding." Crushing tests on 12-in. by 6-in. cylinders gave the following strengths in lb. per square inch : Six days, 4010 ; 27 days, 5705 ; 180 days, 10,600.

The cables and reinforcement of the bottom inertia block are shown in *Fig.* 3. As the baseplates had to be set and levelled precisely, it was not possible for this to be done, and for the top steel to be placed, after concreting the bottom of the block without having a construction joint in the concrete. Similar grout to that used for the concrete was used for grouting the cables as its final setting time of 38 hours and its expansion of 11·6 per cent. by volume after three hours was particularly suitable for this purpose. Before constructing the inertia blocks

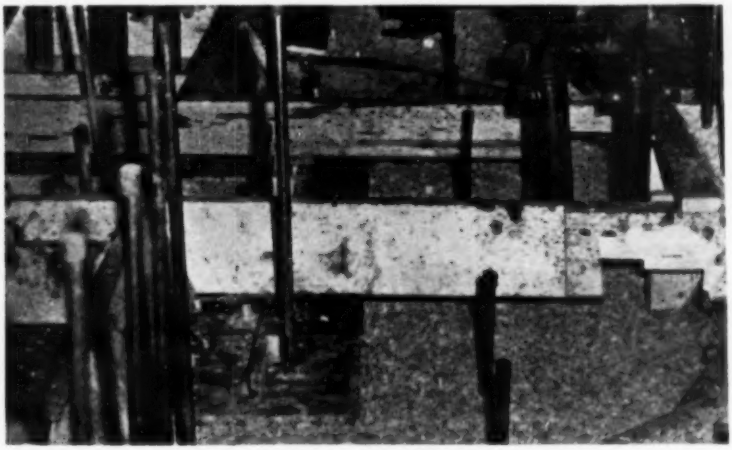


Fig. 4.—Lower Inertia Block with Aggregate in place ready for Grouting.

a model was made showing the reinforcement and cables. This enabled the steel-fixers to plan the work and saved considerable time.

Reconstruction of the foundation pit required about 135 cu. yd. of concrete and presented no particular difficulty other than that of placing concrete amongst closely-spaced reinforcement and using vibrators in confined spaces. After removal of the shuttering it was found that some areas were honeycombed, the most serious being in the piers supporting the dampers at each corner at the bottom level. Here it was necessary to remove about 6 in. from the top of each pier and to cut into the walls alongside the piers in order to form a key between the new concrete and the walls. *Fig.* 3 shows the lower inertia block and the complex reinforcement ; four sets of pipes were provided for draining condensate water through the lower inertia block to the sump. This water also acts as a coolant for the anvil, which becomes very hot. The anchor-cones and the housings for the steel springs were fixed to the shuttering. A view of part of the top of the lower inertia block is shown in *Fig.* 4, with the aggregate and

grouting pipes in position. The grout was pumped into the base of the block and gradually rose through the aggregate. The top surface was shuttered except for the areas occupied by the baseplates shown in *Fig.* 4. This shuttering consisted of parallel boards about ½ in. apart. A layer of expanded metal was laid on top of the boards, followed by a layer of fine wire mesh, the final covering being muslin. This top shutter allowed air and surplus water to escape, while providing a smooth surface.

The wires used in the cables have a carbon content of about 0·7 per cent., a yield stress of about 150,000 lb. per square inch, a tensile strength of about 210,000 lb. per square inch, and a Young's modulus of about 29,000,000 lb. per square inch. It was possible to tension the cables to 140,000 lb. per square inch ; except in the case of one cable which was fixed at both ends by mortar, which entered the sheath during grouting of the aggregate. To allow for the loss of tension in the curved cables—amounting to nearly 40 per cent. in the case of cables A and B—these were generally over-tensioned by about 15 per cent.

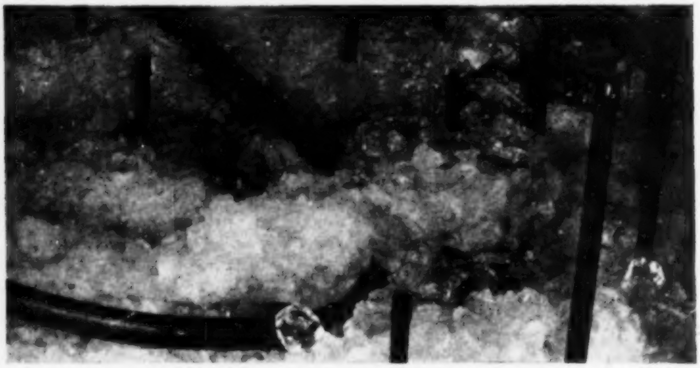


Fig. 5.—Crushed Ice above Aggregate.

On the afternoon before grouting of the lower inertia block started, about 5½ tons of ice were spread over the top of the block and sprinkled with water (*Fig.* 5), the chilled water seeping through the aggregate and reducing its temperature to about 50 deg. F. by the next morning. Grouting started at 8 a.m. and proceeded until about 4 p.m., when the grout was within a few inches of the top. The need to increase the grouting pressure led to some mechanical trouble with the pumping equipment because the pressure required to force the grout under the baseplate was greater than that normally used. Pumping was continued with a standby single-stage pump, and as this was sufficient to supply only one feed-line grouting was not finished until about 10 a.m. the next morning (*Fig.* 6). In order to reduce the rise in temperature of the concrete, water was pumped through the drain-pipes in the central portion of the block throughout the grouting operation and for some time afterwards. Within three hours of completion of grouting, water was sprayed on the surfaces of the block, which were kept wet for several days. The top shuttering was removed on the second day after grouting and the remainder on the fourth day. Tensioning of the

cables was started nine days after grouting, by which time the concrete had a crushing strength greater than 4000 lb. per square inch.

The hammer was placed in service at the end of October, 1951, and has been working since for 24 hours a day, six days a week. The only unsatisfactory grouted concrete was that directly under the anvil baseplate. About two months after the hammer began work, the rubber springs under the anvil appeared to have shifted. The anvil was lifted and the rubber springs and baseplate removed, and it was found that although the surface of the concrete was hard it had in places been depressed by the steel plates slightly more than ½ in. This appeared to have been caused by the presence of froth which seems to be unavoidable

Fig. 6.—Grouting Plant over the Pit. The Pipes Extend Down to the Tops of the Tubes shown in Fig. 3.

in this type of grouted concrete, and is composed of air bubbles in a mixture of grout and water. This froth rapidly broke down under impact and was washed away by water coming through the drains from the condensate. The repair allowed the anvil to be jacked up without dismantling the hammer. The concrete was chipped out to a depth of about 6 in. without disturbing the reinforcement, and the chipped surface was sand-blasted. Using additional reinforcement, a concrete platform 5¼ in. high was constructed to support the baseplate. The mixture used for this concrete was 1 : 1·5 : 1·9 by weight with rapid-hardening Portland cement, and having a compressive strength of 6400 lb. per square inch at 28 days. After three days the surface of this platform was ground to within 0·03 in. of true flatness and treated with a proprietary hardening compound.

Towers for Drying Fire Hoses.

THE towers for drying fire hoses shown in *Fig.* 1 consist of three slender prestressed concrete uprights 30 ft. high with a three-armed precast bracket at the top, from which the hoses are hung. The masts are each made of fourteen precast sections and are arranged to form, on plan, a triangle with sides of 3 ft. Triangular diaphragms, which connect the three uprights at intervals, alternate with these sections and both uprights and diaphragms are pierced with ducts for the passage of Freyssinet prestressing cables. The cable ducts continue into the solid concrete base of the tower and curve outwards at the bottom to allow the cables to emerge from the sides of the base block, where they are tensioned and anchored.

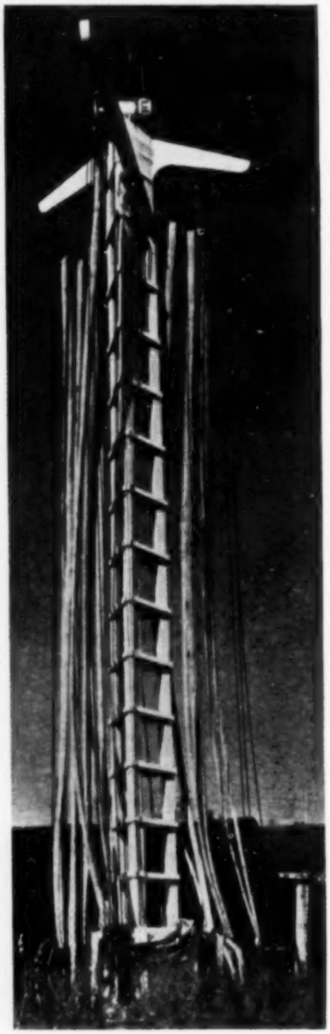


Fig. 1.

A. & C. Buildings, Ltd., who designed the towers in collaboration with the County Architect, Mr. H. Conolly, cast and supplied the parts and carried out the erection and prestressing of a prototype tower. Erection and prestressing of subsequent towers is to be done by the Essex County Council's Fire Brigade Maintenance and Building Department.

Corrosion of Steel in Concrete Pipes.

ACCORDING to "Engineering News-Record" (19 February, 1953) work has been stopped on a contract for making and laying 37 miles of prestressed concrete pipes at Regina, Canada, because it has been found that in some cases the prestressing wire was corroded. The contractors claim that the corrosion is due to the fact that the concrete, which was specified for use in the heavy gumbo clay of the district, contained an admixture to increase the resistance of the concrete to alkalis and calcium chloride to hasten setting. The contractors suggest that these admixtures be omitted and that the pipes be coated with asphalt or other protective cover.

Standard for Slag Aggregates.

THE British Standard for Air-Cooled Blastfurnace Slag Coarse Aggregate for Concrete (No. 1047 : price 4*s*.) has been revised to provide for the use of this class of aggregate in no-fines concrete, and now includes a grading of blastfurnace slag aggregate suitable for no-fines concrete. The methods of taking samples and of carrying out tests have been brought into line with B.S. No. 812, "Methods for the Sampling and Testing of Mineral Aggregates", and the gradings are now the same as for other aggregates.

A Factory for Prestressed Concrete in Argentina.

A LARGE factory for manufacturing prestressed precast concrete products has recently been constructed at Sierra Chica, about 200 miles from Buenos Aires, Argentina. The factory is described in a recent number of the journal "Hormigon Elastico."

The method of manufacture is a long-line process known as the "Zofra" system, designed by M. Zorislav Franjetic, in which a small variety of tubular elements is made which may be assembled to form beams, pipes, posts, and other members.

The arrangement of the factory is shown in *Figs.* 1, 2, and 3. The casting and prestressing beds are in the portion of the factory marked 1 on *Fig.* 1; this portion is 407 ft. long by 108 ft. wide and each of the eight beds is 328 ft. long. The beds are provided with steam pipes for curing the concrete. Above the beds and cantilevered from the columns supporting the roof are platforms extending the whole length of the beds and on which are benches for assembling cages of reinforcement, etc. (*Fig.* 2).

In the part of the factory adjacent to the prestressing beds (marked 2 on *Fig.* 1) the prestressed elements are assembled into the finished products. This portion is 213 ft. long and is divided by columns into nine bays each 27 ft. wide, each bay having a travelling crane. Also in this part are five autoclaves, four 49 ft. long and one 39 ft. long, and two large tanks for curing the products in water.

Adjoining the prestressing bays are the batching and mixing plant and the pumps for distributing the concrete. The aggregate is delivered by rail to the yard adjacent to the batching plant and is then transported by conveyor belts to the screens where it is graded into four sizes.

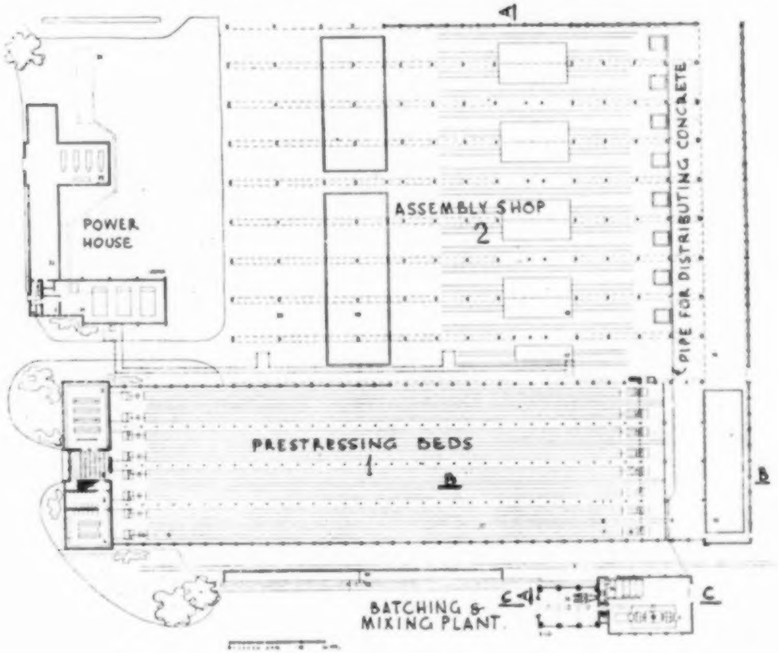


Fig. 1.—Plan.

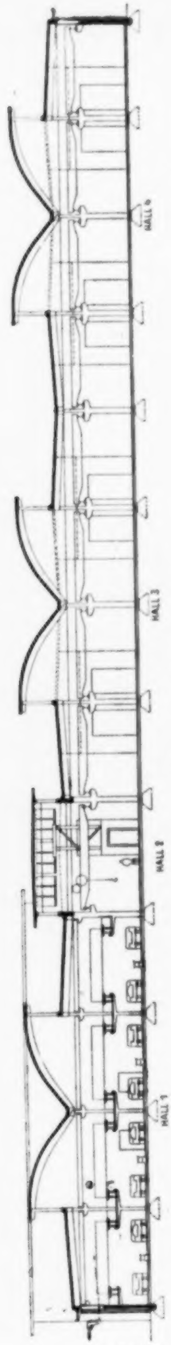


Fig. 2.—Section AA.

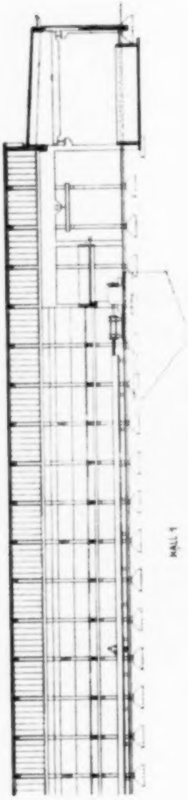


Fig. 3.—Section BB.

Prestressed Concrete Factory in Argentina.

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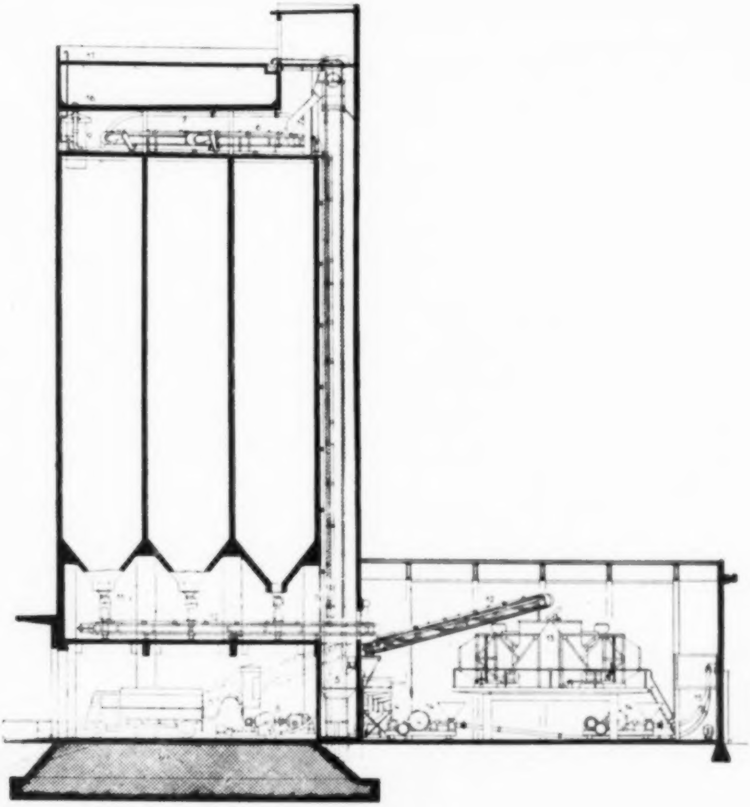

Fig. 4.—Section CC.

The screened material is lifted by elevators to the silos (*Fig.* 4). There are six silos, about 100 ft. high, over the grading plant; four of these are for aggregate and two for cement. On top of the silos is a water storage tank. The dry materials are weigh-batched and carried to the two 2-cu. yd. mixers by belt conveyors. The concrete is distributed to the casting beds and the assembly shop by a concrete pump situated directly below the mixers and a pipe extending along one side of the factory.

The electrical power and the steam required in the factory are produced in the power station to the left of the assembling shop.

The factory is of reinforced concrete and is situated in a village built for the workers. The village contains 310 houses and is provided with an administrative centre, a church, shops, a cinema, a clinic, a swimming pool and a sports ground. The factory was designed by Mr. Franjetic in collaboration with Messrs. Juan Fusek, R. Arendt, and Helmut Kloss. The architect was Mr. Luis M. Stigler.

Prestressed Precast Frame for Elevated Tanks.

THE structure shown in *Fig.* 1 for supporting three elevated tanks, each 30 ft. long by 9 ft. diameter, with a total weight of nearly 150 tons when full, has been completed at the Shell-Mex and B.P. oil installation at Dingle Bank, Liverpool.

The main supports for the tanks comprise sixteen prestressed precast columns, 1 ft. square, set in pockets in reinforced concrete bases. There is no diagonal bracing and the columns resist wind loads as cantilevers about the bases. The tanks are carried on concrete cradles (which were cast at the site) resting on twelve prestressed precast beams 1 ft. 2 in. deep by 10 in. wide which span between the tops of the columns. Prestressed precast beams are also provided between the two outer rows of columns parallel to the length of the tanks to give stability transversely to the main beams. Mild-steel loops protrude from the tops of the columns and the ends of the beams into joints which were filled with concrete to tie the beams and columns together at the level of the platform.

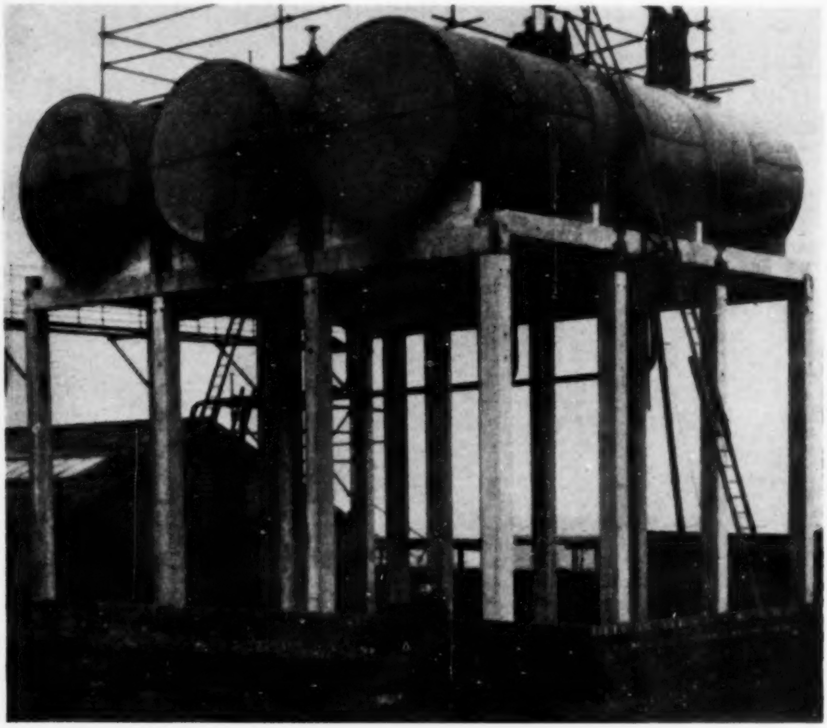

Fig. 1.

The precast columns and beams were designed and made by the Concrete Development Company, Ltd., using 2-mm. wires, on a pre-tensioned long-line process. The concreting at the site and the erection of the structure were carried out by Messrs. Holland & Hannen and Cubitts, Ltd. The total amount of steel used was $1\frac{1}{4}$ tons, and it is stated that had the structure been built in reinforced concrete $18\frac{1}{2}$ tons of steel would have been required.

MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word: minimum 7s. 6d. Situations Vacant, 4d. a word: minimum 10s. Other miscellaneous advertisements, 4d. a word: 10s. minimum. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATION VACANT. Required for consulting engineer's office, experienced detailer for reinforced concrete work. Good draughtsman with drawing-office experience in all types of reinforced concrete, and precast concrete also if possible. Salary according to age and experience. Please write, giving all details, to F. J. SAMUELY, 8 Hamilton Place, London, W.1.

SITUATION VACANT. Consulting structural engineer, Westminster, requires senior designer-draughtsman with first-class experience in reinforced concrete for responsible position. Experience in structural steelwork an advantage. High salary and good prospects for suitable applicant. Write in confidence stating age, qualifications, and full details of experience. Box 3639, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete detailer required. Permanent and progressive post. Some previous experience and good drawing essential, but opportunity to obtain experience in prestressed and other structural design. Write DONOVAN LEE, M.I.C.E., Consulting Engineer, 66 Victoria Street, London, S.W.1.

SITUATIONS VACANT. Designer-detailers required by consulting engineers for their Newcastle-upon-Tyne office. Applicants should have had previous experience of heavy reinforced concrete industrial foundations, structures, and other civil engineering works. Appointments offer chances to secure sound and extensive experience, and will carry good salaries and prospects according to experience and ability. Apply, giving full particulars of qualifications, training, and experience, together with a statement as to salary required, to Box 3645, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Engineering draughtsman required at Feltham, Middlesex, with commercial experience in reinforced concrete design, especially floors. Write experience and salary required to Box 3644, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, LONDON, S.W.1.

SITUATIONS VACANT. Imperial Chemical Industries Limited, General Chemicals Division, require general civil and structural designer-draughtsmen to assist in the design of steel and reinforced concrete structures for chemical plants. Location: Runcorn. Salary dependent on age and experience. Apply in writing, quoting E/83, to STAFF MANAGER, IMPERIAL CHEMICAL INDUSTRIES LIMITED, General Chemicals Division, Cunard Building, Liverpool, 3.

SITUATION VACANT. Senior reinforced concrete engineer required for Midland office of reinforced concrete specialists. A qualified man of wide experience is required, capable of controlling drawing office staff. Salary according to ability but not less than £1,000 per annum. Apply Box 739, 19/21 Corporation Street, Birmingham 2.

SITUATION VACANT. Designer-draughtsman required by specialist manufacturers of steel shuttering and moulds. Knowledge of these matters preferable. Good prospects for right man. Salary according to experience. Box 3646, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. THE TRUSSED CONCRETE STEEL CO., LTD., Truscon House, 35-41 Lower Marsh, London, S.E.1, have a vacancy in their London office for a measurer with experience in the taking-off and preparation of bills of quantities mainly for reinforced concrete structures. Age 25-30. Five-days' week and pension scheme. Apply in writing to the above address, giving full particulars of age, education, and previous employment.

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SITUATION VACANT. Planning assistant with previous experience in costing, estimating, or progress work in building, civil engineering, precast concrete, or allied industries. This vacancy offers excellent scope for a young man of about 25-30 who has ability and can work. Apply BEECHAM BUILDINGS, LTD., Reinforced Concrete Engineers, Shipston-on-Stour, Warwickshire.

SITUATIONS VACANT. THE TRUSSED CONCRETE STEEL CO., LTD., Truscon House, 35-41 Lower Marsh, London, S.E.1, have vacancies in their London, Birmingham, Glasgow, and Manchester offices for designers and designer-detailers with considerable experience in reinforced concrete D.O. work. Five-days' week and pension scheme. Apply in writing to the above address, giving full particulars of age, education and previous employment.

SITUATIONS VACANT. NORTH THAMES GAS BOARD.—The following Drawing Office staff are required in the Chief Engineer's department. (i) At Beckton, E.6, Bromley-by-Bow, E.3, Lea Bridge, E.10, Mill Hill, N.W.7, and Kensal Green, W.10—Draughtsmen experienced in the maintenance of gas manufacturing and ancillary plant. Starting salary within the range £370-£600 per annum. (ii) At Westminster, S.W.1—(a) Senior Draughtsmen experienced in the design of gas manufacturing and ancillary plant, including steel-frame structures. Starting salary within the range £645-£795 per annum. (b) Draughtsmen experienced in the layout and detailing of gas manufacturing and ancillary plant, including steel-frame structures. Starting salary within the range £520-£695 per annum. (c) Draughtsmen for detailing reinforced concrete structures. Experience in design would be an added advantage. Starting salary within the range £520-£695 per annum. All these appointments are of a permanent nature, and successful candidates will be required to join the staff pension scheme. Starting salaries within the ranges mentioned will be dependent on age and qualifications. Applications, giving full particulars, should be sent to the STAFF CONTROLLER, NORTH THAMES GAS BOARD, 30 Kensington Church Street, London, W.8, to reach him not later than ten days after the publication of this advertisement and quoting reference 666/63.

(Continued on p. 157.)

(*Continued from p. lii.*)

SITUATION VACANT. Position available for technical assistant works manager with experience in prestressed and reinforced concrete. Attractive prospects for suitable candidate. Technical qualifications not essential but preferred. Apply, with full particulars of experience and salary required, to ANGLIAN BUILDING PRODUCTS, LTD., Atlas Works, Lenwade, Norwich.

SITUATION VACANT. Old-established Tees-side firm requires section-leader reinforced concrete designer-draughtsman, fully experienced in designing and detailing reinforced concrete structures, foundations, and other civil engineering work. Apply, giving full particulars and experience, quoting D, to Box 3647, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete engineers require designer-draughtsmen, preferably A.M.I.Struct.E., for work at Bristol or near London. Opportunity to widen experience. Good salary to right men. Box 3648, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Consulting engineer requires senior reinforced concrete designer to take charge of design and administration of contract in drawing office. Position offers scope for organising ability, and some previous site experience and knowledge of engineers' quantities would be an advantage. Applicants should be Corporate Members of the Institution of Structural Engineers, or have equivalent experience. Write, stating age, experience, and salary required, to Box 3649, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. The services of a further detailer-draughtsman are required by consulting engineers who have filled previously-advertised situations and are further expanding. The work covers mainly structures in connection with coal-preparation plants. The position available is a progressive one and has possibilities for a "live" young man who is keen to get experience. Applicants should be preferably to H.N.C. standard and have Institution of Structural or Civil Engineers examinations in mind. 5-days' week. Write, stating age, experience, and salary required, to J. C. HUGHES & PARTNERS, 119 Marylebone Road, London, N.W.1.

SITUATION VACANT. Technical assistant required at concrete depot in Manchester area by British Railways. Must have ability to control staff, plan output, and prepare formwork details. Knowledge of elements of reinforced and prestressed concrete desirable. Commencing salary £612 5s. per annum. Certain residential and free travelling facilities given. Apply CIVIL ENGINEER, BRITISH RAILWAYS, LONDON MIDLAND REGION, Euston Grove, London, N.W.1.

SITUATIONS VACANT. Structural engineering designer-draughtsmen required in London in Designs Branch of Air Ministry Works Department. Applicants should have several years' experience in design and detailing of reinforced concrete or structural steelwork. Salaries in ranges up to £733 per annum, starting pay dependent upon age, qualifications, and experience. Overtime and extra duty allowance payable. Apply, quoting Order No. Borough 3674/MA., stating age, qualifications, and previous appointments (with dates), to any Employment Exchange.

SITUATION VACANT. Reinforced concrete designer-draughtsman for consulting engineer's office, North London. Good salary and good prospects. Interesting all-round work. Apply to L. J. ELGIN, A.M.I.C.E., 47 Clarendon Court, London, N.W.11. Telephone: Mountview 5083.

SITUATION VACANT. Required for consulting engineer's office, tracer with a minimum of two years' experience in structural drawing. Salary according to age and experience. Please apply in writing, giving all details, to F. J. SAMUELY, 8 Hamilton Place, London, W.1.

SITUATIONS VACANT. Ove Arup & Partners require several reinforced concrete designer-detailers. Apply in writing giving full particulars of education, training, and experience, and stating salary required, to above at No. 8 Fitzroy Street, London, W.1.

(*Continued in next column.*)

Books Received.

"**The Study of Road Subgrades.**" By A. E. Oliveira. (Lisbon: Ministry of Public Works. 1951. No price stated.)

"**Medição de Deformações com Extensometros Mecânicos.**" By J. D'Arga e Lima. (Lisbon: Ministry of Public Works. 1951. No price stated.)

"**Building Construction.**" **Volume I.** By W. B. McKay. Third Edition. (London: Longmans, Green & Co. 1953. Price 12s. 6d.)

"**Principles and Practice of Prestressed Concrete.**" Vol. 1. Second edition. By P. W. Abeles. (London: Crosby Lockwood & Son, Ltd. 1952. Price 21s.)

"**A Survey of the Behaviour in use of Asbestos-Cement Pressure Pipes.**" By F. E. Jones and J. P. Latham. (H.M. Stationery Office. Price 2s. 6d.)

"**Structural Steelwork for Buildings.**" By H. P. Smith. Revised edition. 1952. (London: Crosby Lockwood & Son, Ltd.)

(*Continued from column 1.*)

SITUATION VACANT. Resident engineer required to supervise construction of large reinforced concrete office building in London, W.C.1. A chartered civil or structural engineer preferred. Must be experienced in supervising first-class work. Reply giving full details of experience and stating salary required to Box 3650, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Designer-draughtsman required by Westminster consulting engineer, preferably with experience of river and dock work. Apply giving full particulars, and salary required, to T. F. BURNS & PARTNERS, 25 Victoria Street, London, S.W.1.

SITUATION VACANT. Concrete research engineer. The Stanton Ironworks Co., Ltd., proposes to appoint a young man with civil engineering experience prepared to take a keen interest in concrete research and quality control in respect of normally reinforced and prestressed concrete. The appointment carries excellent prospects, and the post is superannuated. Write in confidence to the STAFFING OFFICER, THE STANTON IRONWORKS CO., LTD., near Nottingham, stating in detail age, qualifications, and experience.

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers and detailers with specialist experience for their Stafford, Bristol, Glasgow, London, Newcastle-upon-Tyne, and Dublin offices. Five-days' week, and staff pension scheme. Apply to B.R.C. ENGINEERING CO., LTD., Stafford.

PROFESSIONAL SERVICES.

PROFESSIONAL SERVICES. Reinforced concrete designer-draughtsman with 20 years' experience in the design of reinforced concrete structures, foundations for heavy industrial structures, chemical plant, and oil refineries, undertakes to prepare complete designs, calculations, and working details to help consulting engineers. Excellent references, moderate fee. Box 3637, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

FOR HIRE.

FOR HIRE. Lattice steel erection masts (light and heavy), 30 ft. to 150 ft. high, for immediate hire. BELLMAN'S, Terminal House, London, S.W.1. Telephone: Sloane 5259.

FOR SALE.

FOR SALE. Sacks, bags, and curing cloths for sale. You want the best type and quickest delivery. Write JOHN BRAYDON, LTD., 26 The Highway, London, E.1. Telephone: ROYal 1044.

FOR SALE. Steel angle fencing stakes. All sizes in stock. List on request. E. STEPHENS & SON, LTD., Bath Street, London, E.C.1.

FOR SALE. Concrete mixer. Capacity 10/7 with Bamford engine. F. J. EDWARDS, LTD., 359 Euston Road, London. Telephone: EUSton 4681.

Prefabricated Reinforcement.

THE framework of reinforcement shown in *Fig.* 1 has been designed and erected in accordance with a system recently introduced which, it is stated, reduces the time required for the fixing of reinforcement on the site to one-twelfth of that needed by the usual methods. It is also claimed that the weight of steel required is reduced by the elimination of hooks or other anchors at the ends of the bars where these have stirrups welded to them.

The method, which is known as the "Frameweld" system, consists of the prefabrication in a workshop of welded cages of reinforcement which are erected on the site in the manner of structural steelwork. The columns comprise main bars with a continuous binding welded at each intersection with the main bars. The columns are provided with 1-in. by ¼-in. mild steel flats welded to the main bars to support the reinforcement of the beams (*Fig.* 2). The top edges of the flats may be grooved to receive these bars and ensure that they are in their correct position. The beam reinforcement is connected to the supporting flats by ¼-in. diameter hook-bolts passing over the tops of the bars and through mild steel flats under the supporting flats. Alternatively, pieces of 10-gauge wire 3 in. or 4 in. long may be welded to the flats before these are fixed in position and the ends of the wires turned around the bars as these are placed.

Connections between consecutive lifts of column bars are made by welding 1-in. by ¼-in. flats to the lower column bars at the level of the bottom of the crank where the main bars are lapped and to the bars of the upper column about 1 in. from their lower ends. The bars in the upper column are then supported by two more flats placed across those of the lower column and attached to them by being welded to washers 1 in. square kept in position by hook-bolts on the lower flats.

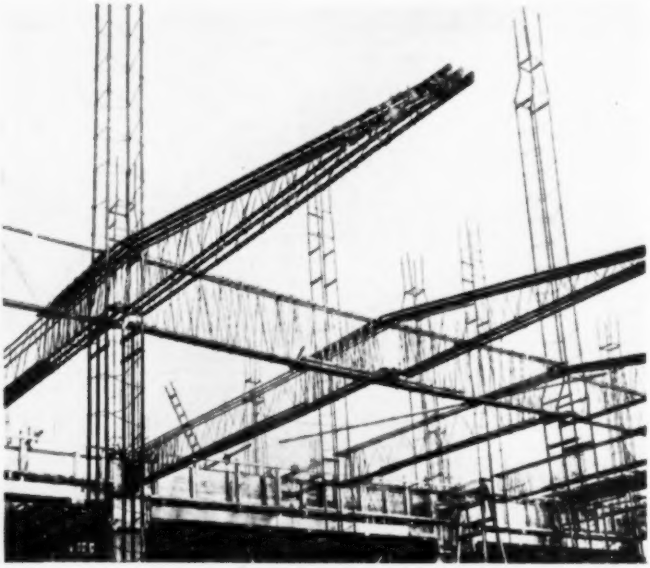
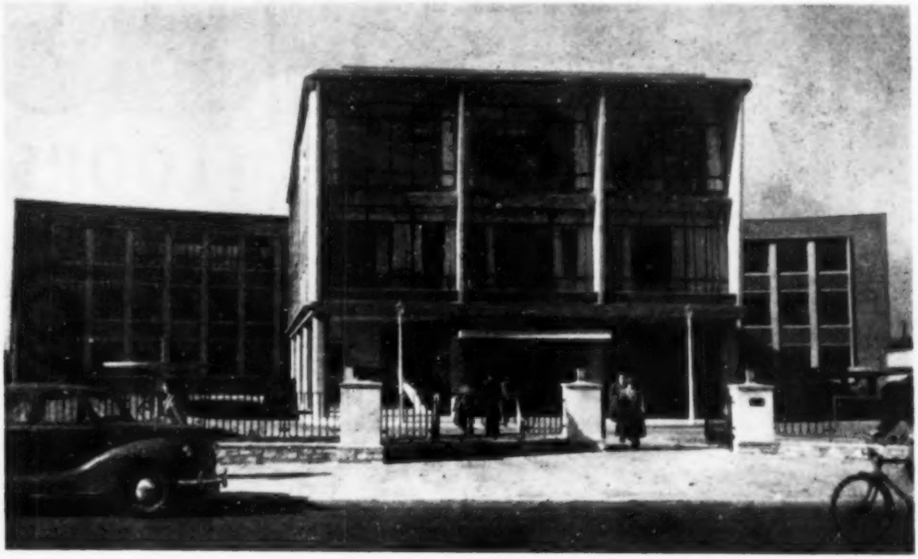


Fig. 1.—Reinforcement Erected.

The beam reinforcement comprises a number of separate units each consisting of upper and lower bars joined by a diagonally-inclined continuous stirrup welded to the longitudinal bars. Trans-



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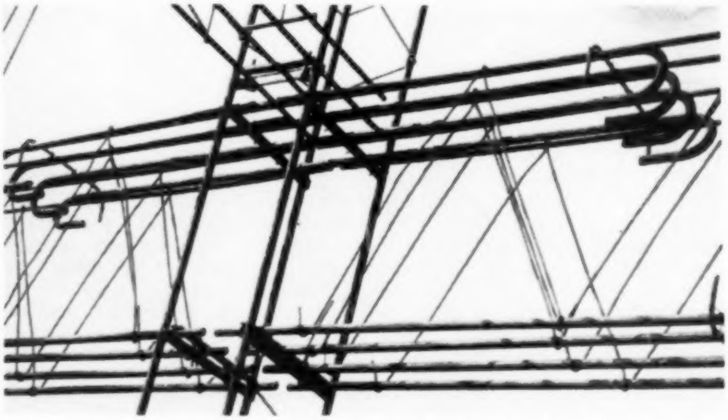

Fig. 2.—Supports on the Column Bars for Beam Reinforcement.

verse stirrups are not used, and this enables the concrete to be more easily compacted. Distance-pieces are used to maintain the spacing of the reinforcement in the shutters. The standard units have stirrups inclined at 60 deg. or 45 deg., but additional shear reinforcement can be provided by increasing the angle of inclination of the stirrups or their diameter (*Fig.* 3). If more than one layer of bars is required in any portion of the beam the additional reinforcement is welded to the stirrups when fabricating the cage (*Fig.* 3).

Reinforcement over the supports of continuous beams is not made in cage form as this would lead to difficulty in fixing. The bars are supplied loose and supported on flats welded to the column bars as well as being tied to the top bars of the beam reinforcement (*Fig.* 2). Transverse ties are used at intervals to keep the bars properly spaced.

The shuttering may be supported by props in the usual manner or be carried by steel channels bearing on the concrete of columns already cast.

In order to determine the effectiveness of the reinforcement when hooks, transverse stirrups, and diagonal bars were

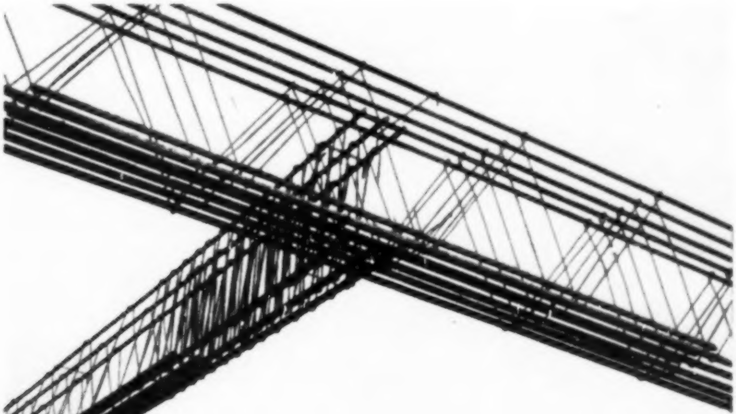

Fig. 3.—Additional Main Bars and Stirrups.

omitted, tests were made at the Building Research Station of twenty-four beams, eight reinforced in the ordinary way and sixteen with welded frames of the form described. The beams were 12 in. deep by 6 in. wide. One-half of the beams in each group was 10 ft. 6 in. long and the others 5 ft. 6 in. The beams were loaded at one-third points, and over a span of 10 ft. there was no significant difference in the strength or deflection of the two types of beams. The beams with the welded cages of reinforcement had no shear reinforcement additional to the diagonally-inclined stirrups, and consequently on the shorter span they failed under a smaller load than the normally-reinforced beams provided with shear reinforcement. The tests showed that the bond strength of bars with stirrups welded to them was as great as that of hooked bars with loose stirrups.

The system has been designed by Messrs. T. C. Jones & Co., Ltd., and used by them for several structures including extensions of factories, a seven-story office building in London, and new laboratories at St. George's Hospital, London. The system has also been used in the construction of raft foundations on a housing site in Wales.

Stable Concrete Mixtures

In Bulletin No. 24 of the Swedish Cement and Concrete Research Institute a report, in English, is given of investigations of stable concrete mixtures. This term is used to describe mixtures the deformability of which remains constant during a reasonable period of consolidation. If the deformability decreases as the period of consolidation increases segregation occurs, while a marked increase in deformability indicates poor workability with the risk of inadequate compaction. Various methods of stabilising a mixture are discussed, including the effects of the addition of resins, and are illustrated by examples from practice.

A device for measuring the deformability has been developed in which the free natural oscillation of a system is damped by the concrete, the damping increasing as the deformability becomes lower. The oscillating system (*Fig.* 1) consists of a vertical torsional shaft (1) the upper end of which is clamped to the frame and the lower end coupled to the concrete by means of the device (2). The concrete is filled into the container (3) and bolted to the vibrator (5). The frequency of

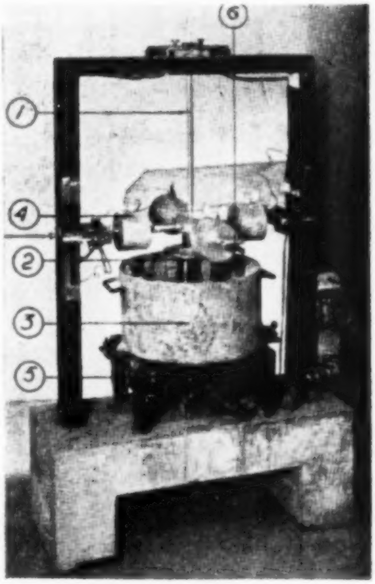

Fig. 1.

oscillation of the shaft is controlled by the weights (4) attached to the cross-arms of the shaft. The measurements can be made during vibration or while the concrete is at rest. The oscillation is initiated by an instantaneous impulse produced by discharging capacitor batteries through electromagnets fixed at the ends of one of the cross-arms and is recorded by the plate (6) which screens off part of a cone of rays from a projector before the rays reach a photo-electric cell. The variations in the current produced by the cell are transmitted to an oscillograph and are recorded on a film.

DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(*Delivered in London area.*)

AGGREGATES (per cu. yd.). — Washed sand, 21s. 9*d*. Clean shingle: ¾ in., 18s. 5*d*.; ⅜ in., 21s. Pit ballast, 20s. 11*d*.

CEMENT (per ton, delivered at Charing Cross).—
Portland cement, 6 tons and upwards, 91s. 1 ton to 6 tons, 102s. Paper bags and non-returnable jute sacks included.
Rapid-hardening Portland, 8s. above ordinary Portland.
Aquacrete and 417, 32s. 6*d*. above ordinary Portland; paper bags included.
Colorcrete (buff, red, and khaki), in 6-ton loads, 134s. 6*d*.; paper bags included.
Snowcrete, £12 13s. 6*d*., inc. paper bags.
"Super-Cement," 32s. 6*d*. per ton above ordinary Portland cement; paper bags included.
High-alumina cement, 1 ton and upwards, 280s. per ton.
Snowcem paint, 71s. per cwt. inc. containers.

SHUTTERING.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1s. 1*d*.) and No. 94 (price 5*d*.) issued by H.M. Stationery Office.

REINFORCEMENT.—Mild steel bars, B.S. 785 (per cwt.); ⅝ in. to 2¾ in., 40s. 5/16 in. to ½ in., 41s. 6*d*. ⅜ in., 42s. ¼ in., 43s. 6*d*.

Materials and Labour.

(*Contracts up to £5000. Inc. 10 per cent. profit.*)

PORTLAND CEMENT CONCRETE, 1 : 2 : 4.—
Foundations, 2s. 3½*d*. per cu. ft. Columns, 3s. 2*d*. per cu. ft. Beams, 2s. 11*d*. per cu. ft. Floor slabs 4 in. thick, 8s. 4½*d*. per sq. yd.; Do., 5 in., 10s. 6*d*.; Do., 6 in., 12s.; Do., 7 in., 14s. 1*d*. Walls 6 in. thick, 13s. 5½*d*. per sq. yd. Add for hoisting above ground level 3s. 7*d*. per cu. yd. Add for rapid-hardening Portland cement 2s. 6*d*. per cu. yd.

REINFORCEMENT.—Mild steel bars (B.S. 785), including cutting, bending, fixing, and wire (per cwt.)—¼ in. to ⅜ in., 70s. 3*d*. 7/16 in. to ⅝ in., 64s. 6*d*. 11/16 in. to 2¾ in., 59s.

SHUTTERING AND SUPPORTS.—
Walls, 201s. per square.
Floors (average 10 ft. high), 186s. 6*d*. per square. In small quantities, 2s. 5½*d*. per sq. ft.
Columns, average 18 in. by 18 in. (per sq. ft.), 2s. 10*d*.; in narrow widths, 3s. 7*d*.
Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 2s. 9*d*.; in narrow widths, 3s. 3*d*.
Raking, cutting, and waste, 5½*d*. per lin. ft.
Labour on splays, 2¾*d*. per lin. ft.
Small fillets to form chamfers, 5½*d*. per lin. ft.

Wages.

The rates of wages on which the above prices are based are: Carpenters and joiners, 3s. 8*d*. per hour (carpenters 2*d*. a day tool money); Labourers, 3s. 2½*d*.; Men on mixers and hoists, 3s. 4½*d*.; Bar-benders, 3s. 5*d*.

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